CHAPTER 12
CONCRETE BENT CAPS

TABLE OF CONTENTS

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CHAPTER 12
CONCRETE BENT CAPS

12.1 INTRODUCTION

A bent consisting of columns and a bent cap beam is an intermediate support between bridge spans that transfers and resists vertical loads and lateral loads such as earthquake and wind from the superstructure to the foundation. The bent cap beam supports the longitudinal girders and transfers the loads to the bent columns. Concrete bent cap beams may be cast-in-place or precast and may be either conventionally reinforced or prestressed.

A typical elevation view of a concrete bent integrally connected with the superstructure is shown in Figure 12.1-1.

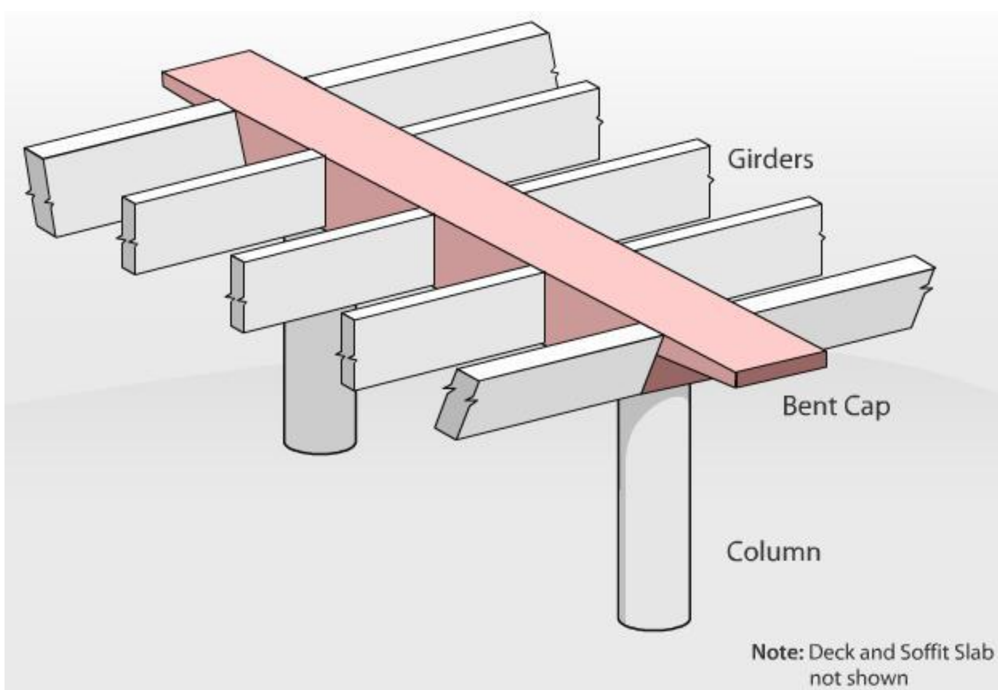


Figure 12.1-1 A Typical Integral Concrete Bent

Bents can be classified as a single-column, a two-column, or a multicolumn bent as shown in Figure 12.1-2.

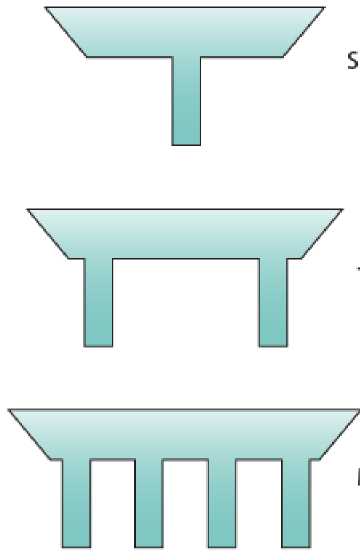


Figure 12.1-2 Typical Bents

12.1.1 Types of Bent Caps

The main types of bent caps are:

- Drop bent cap
- Integral bent cap
- Inverted tee cap

These bent caps may be configured in conventional bent types as shown in Figure 12.1-2, and may possess asymmetric column configurations. Also, they may be utilized in unusual bent types, such as "C" bents, and outrigger bents.

12.1.1.1 Drop Bent Cap

A drop bent cap, as shown in Figure 12.1-3, supports the superstructure girders directly on its top. This type of bent cap is generally used when the superstructure consists of precast concrete or steel girders.

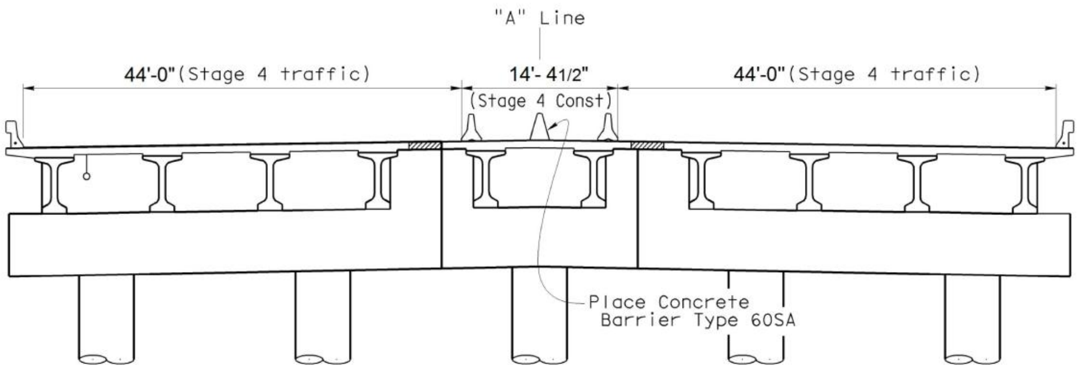


Figure 12.1-3 Overview of Drop Bent Cap

Drop bent caps may have different types of connection to the superstructure diaphragm: fixed, pinned, or isolated. Figures 12.1-4 to 12.1-6 show each type of bent cap.

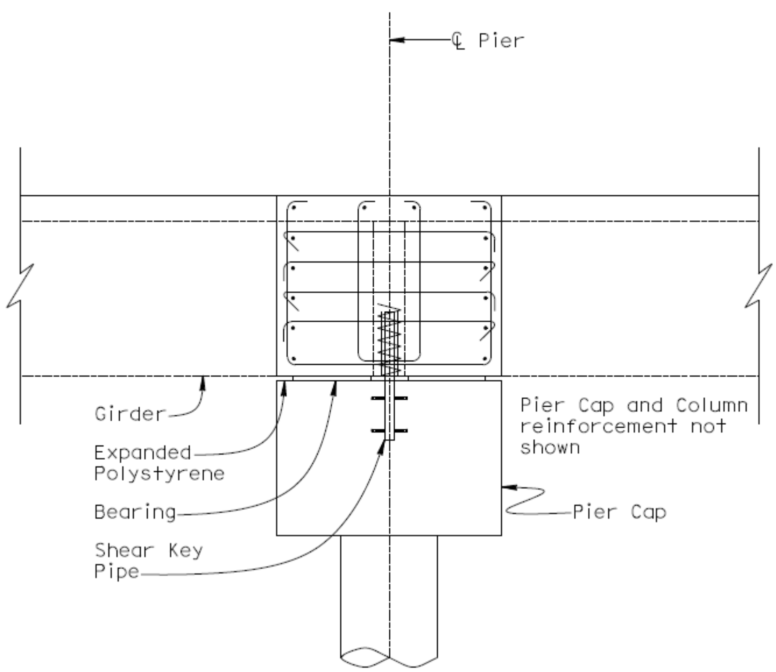


Figure 12.1-4 Drop Cap with Pinned Connection

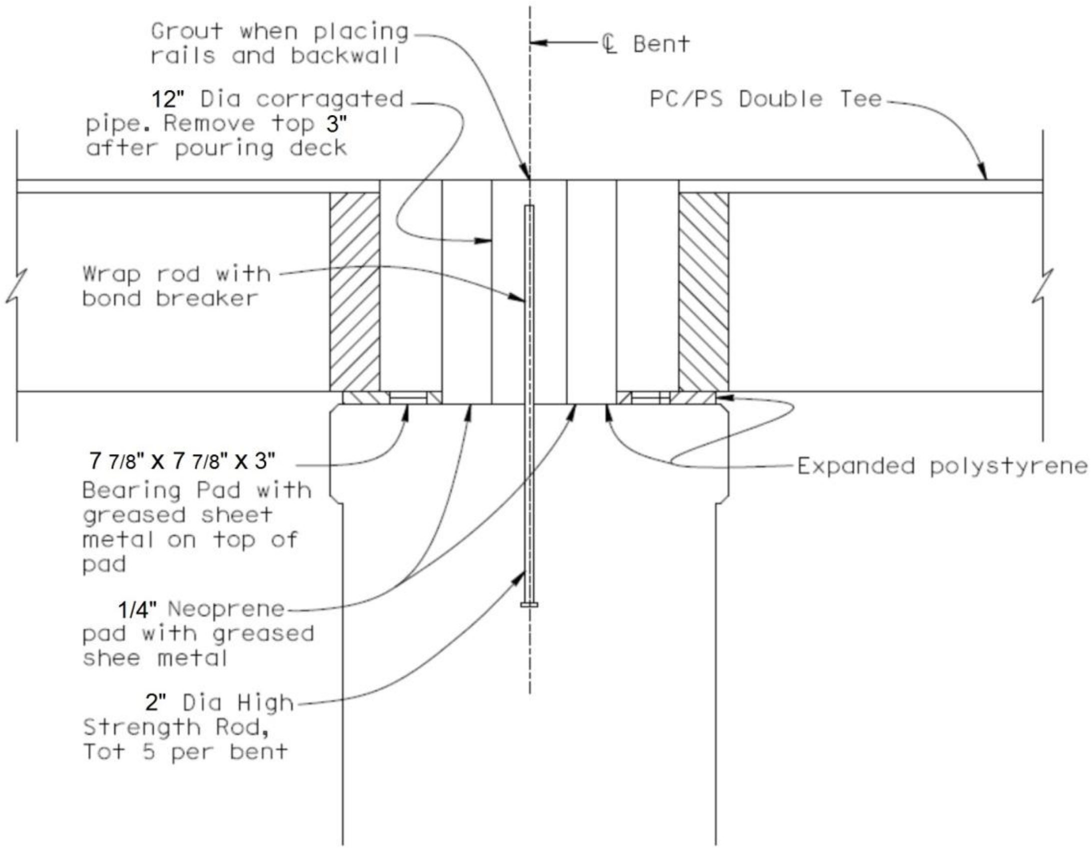


Figure 12.1-5 Drop Cap with Isolated Connection

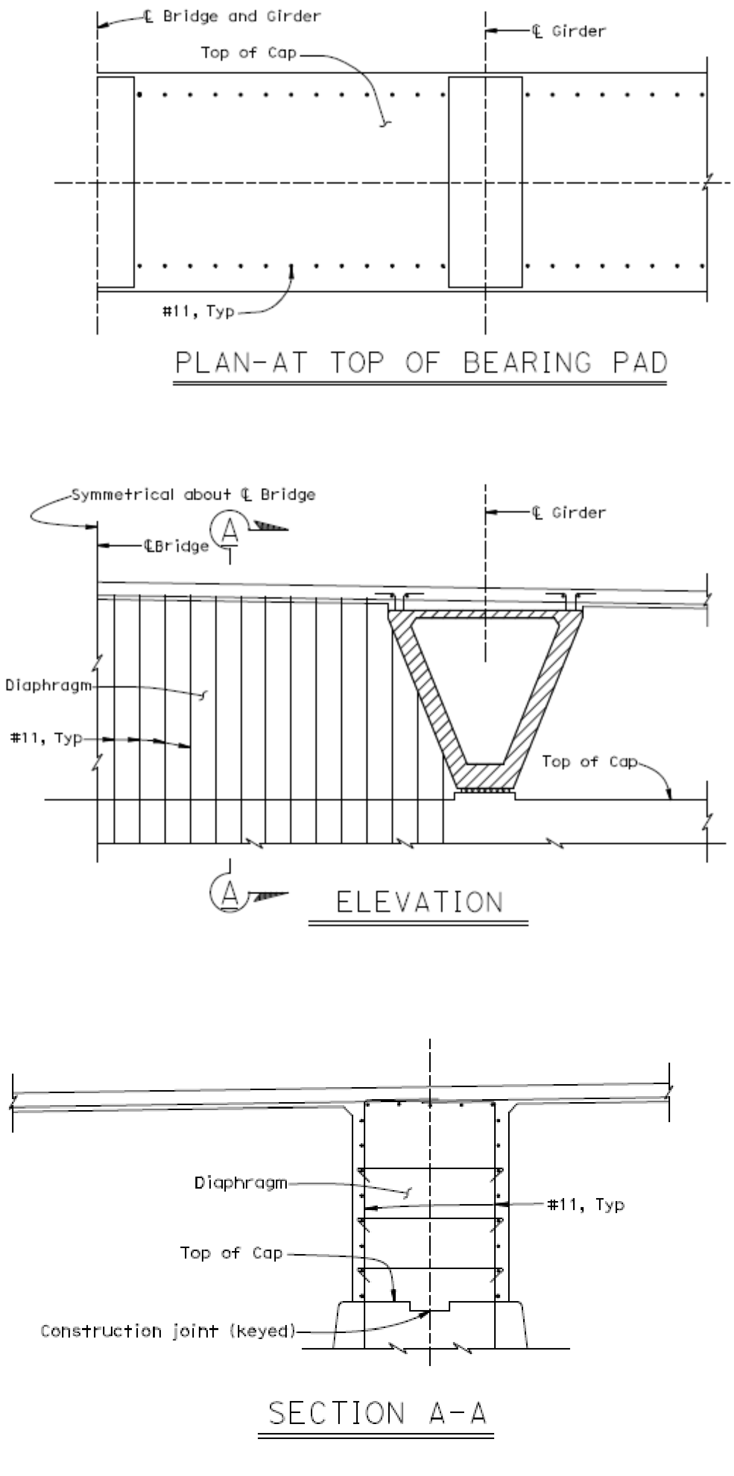


Figure 12.1-6 Drop Cap with Fixed Connection

12.1.1.2 Integral Bent Cap

An integral bent cap, shown in Figure 12.1-7, is cast monolithically with the superstructure girders, and typically has the same depth as the superstructure. The superstructure girders are framed into the bent cap and are supported indirectly by the bent cap. This type of bent cap is commonly used in cast-in-place concrete box girder construction. The load from the girders is transmitted as point loads along the length of the bent cap.

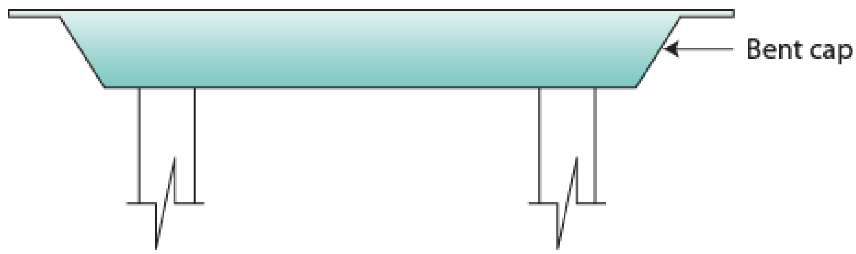


Figure 12.1-7 Integral Bent Cap

As a monolithic connecting element to columns and girders, reinforcement details in integral bent caps can be challenging. Figures 12.1-8 to 12.1-12 show three-dimensional schematics of bar reinforcement belonging to components from the superstructure that must be accommodated by the integral bent cap. Engineers must also consider the integration of bar reinforcement from the columns.

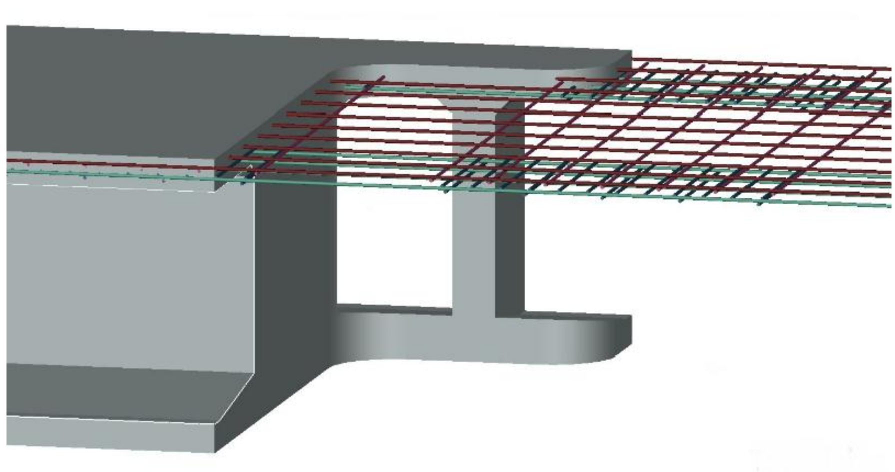


Figure 12.1-8 Integral Bent Cap Top Slab Reinforcement

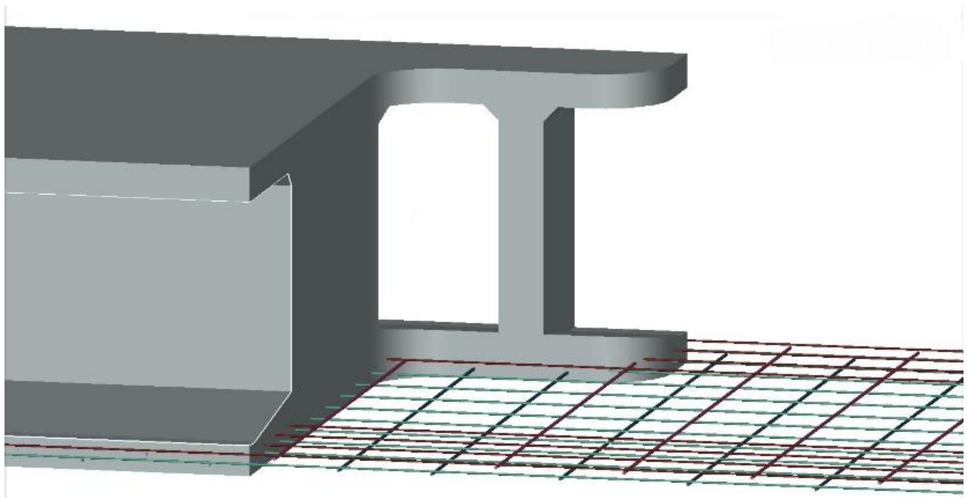


Figure 12.1-9 Integral Bent Cap Bottom Slab Reinforcement

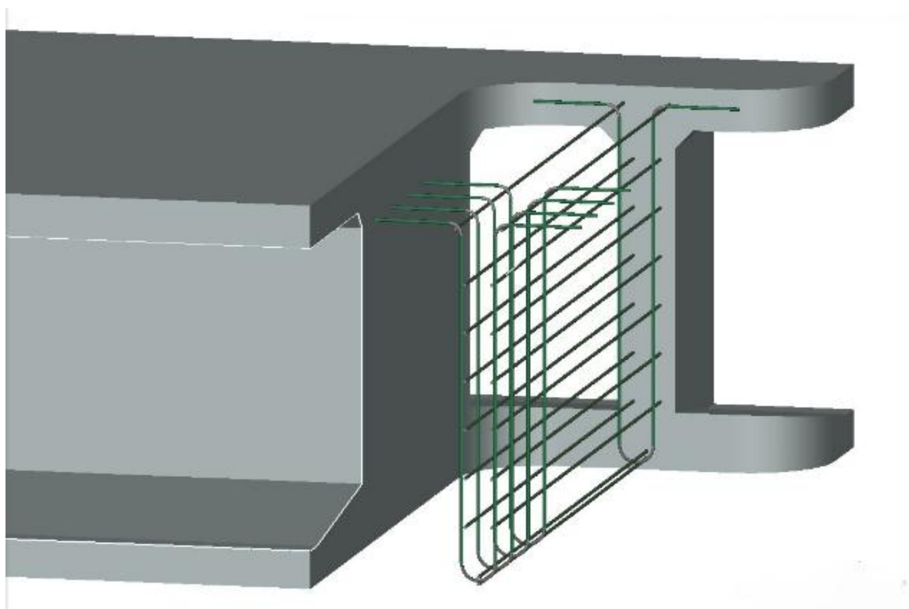


Figure 12.1-10 Integral Bent Cap Girder Reinforcement

Note: For clarity, post-tensioning ducts are not shown.

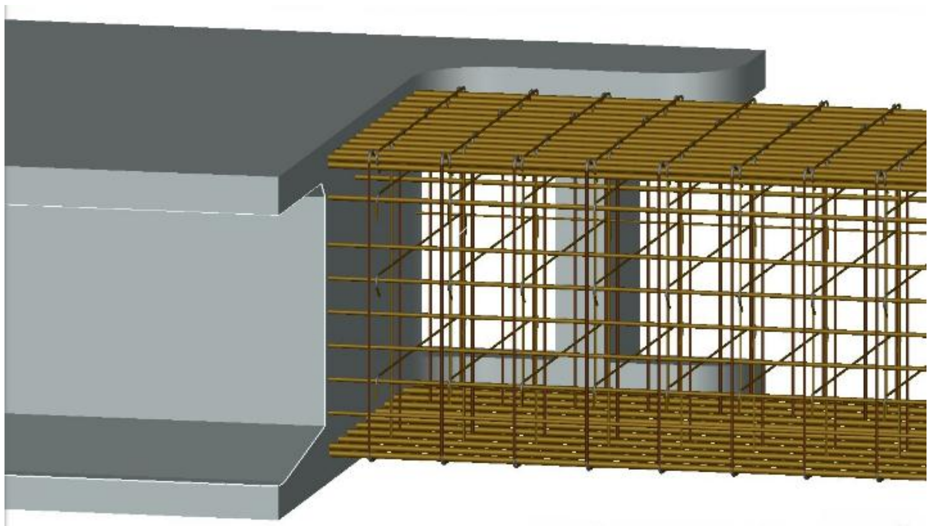


Figure 12.1-11 Bent Cap Reinforcement of Integral Bent Cap

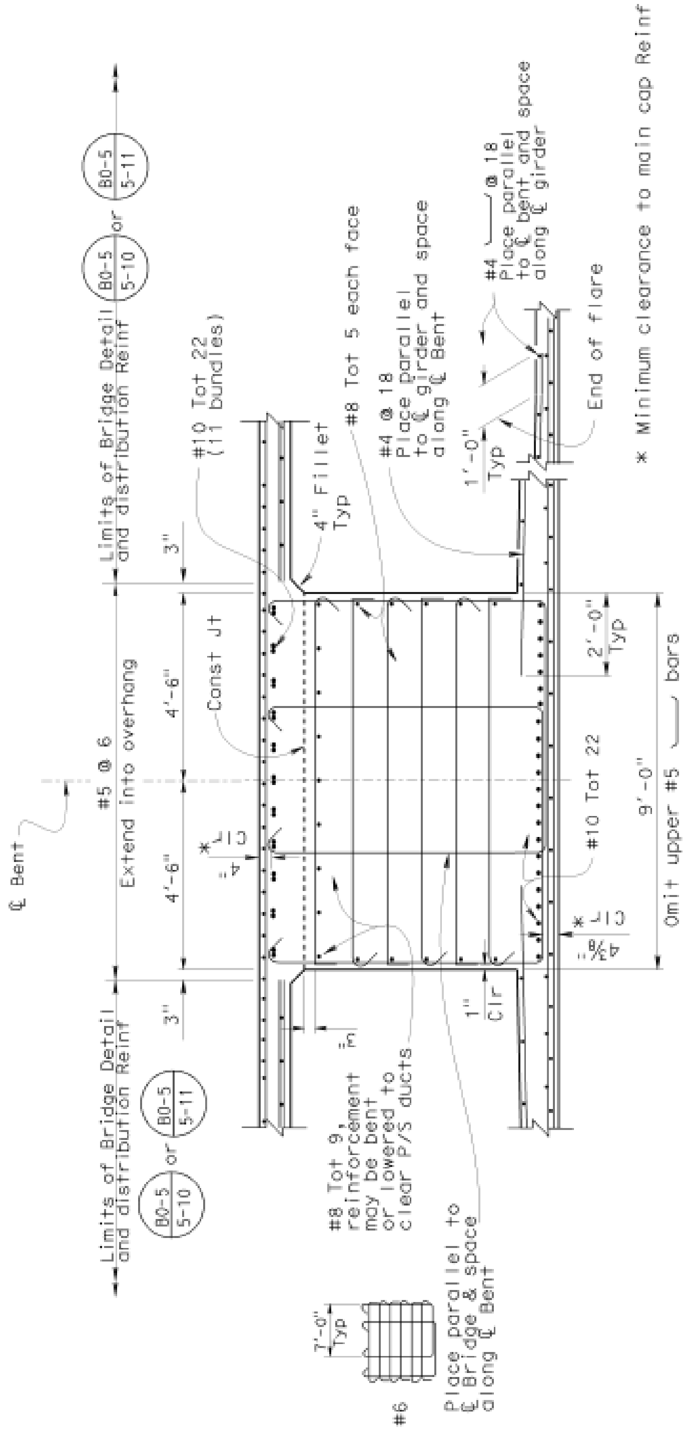
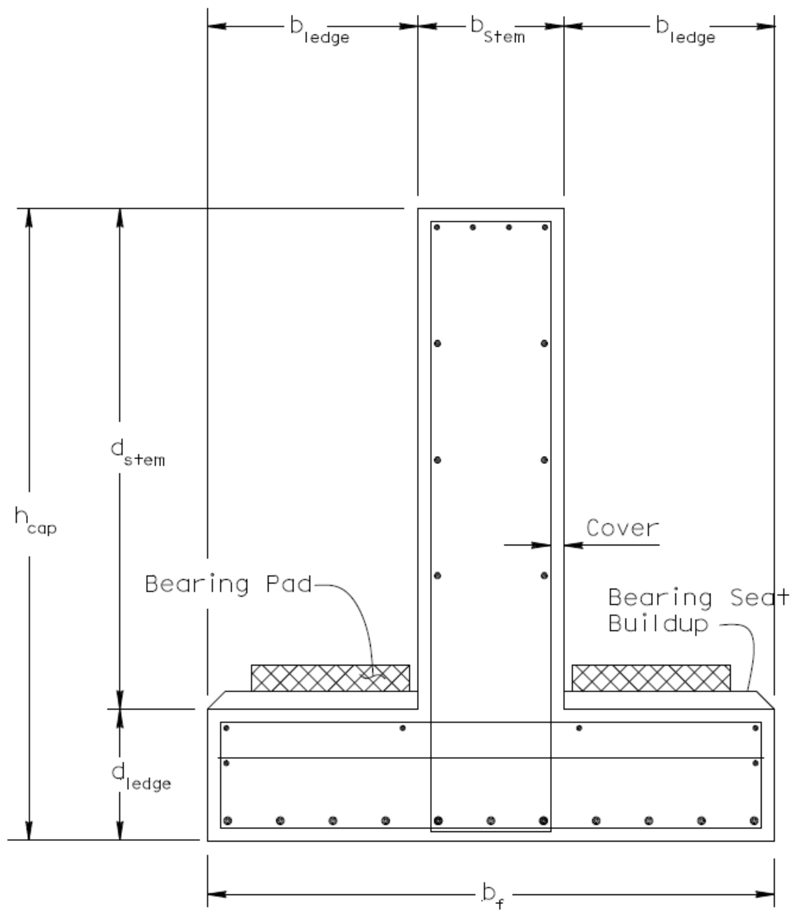


Figure 12.1-12 Integral Bent Cap Cross Section

12.1.1.3 Inverted Tee Cap

Inverted tee cap, as shown in Figure 12.1-13, is typically used with precast concrete girders to increase vertical clearance and to enhance aesthetic appearance. However, from a design standpoint, it is difficult to satisfy seismic demands, and the reinforcement of the ledge of the tee cap presents special challenges in shear, flexure, and bar anchorage.



b_{ledge}	= ledge width	d_{ledge}	= ledge depth
b_{stem}	= stem width	d_{stem}	= stem depth
b_f	= flange width	h_{cap}	= bent cap depth

Figure 12.1-13 Inverted Tee Bent Cap

12.1.2 Member Proportioning

The bent cap depth should be deep enough to develop the column longitudinal reinforcement without hooks in accordance with SDC 7.3.4 and 8.2.1 (Caltrans, 2013). For integral bent caps, the minimum bent cap width required by SDC 7.4.2.1 (Caltrans, 2013) for adequate joint shear transfer shall be the column sectional width in the direction of interest, plus two feet.

Drop caps that support superstructures with expansion joints must have sufficient width to prevent unseating. In accordance to SDC 7.3.2.1 (Caltrans, 2013), the minimum width for non-integrated bent caps is determined by considering displacement of the superstructure due to prestress shortening, creep and shrinkage, thermal expansion and contraction, and earthquake displacement demand.

For inverted tee bent caps, the stem must be a minimum three inches wider than the column to allow for the extension of column reinforcement into the cap. Similar to the integral and drop caps, the depth of the inverted tee cap must be adequate to develop the column bar reinforcement without hooks. The ledge width (b_{ledge}) must be adequate for the primary flexural reinforcement to develop fully.

A bent cap may be evaluated as a "conventional beam" or a "deep beam" in accordance with AASHTO LRFD Article 5.6.3.1 (AASHTO, 2012) to estimate the internal forces of the bent cap. AASHTO Articles 5.6.3.1 and 5.8.1.1 specify that if either of following two cases is satisfied, then a strut-and-tie model may be used:

Case 1:

$L_{v,zero} < 2d$

where:

$L_{v,zero}$ = distance from the point of zero shear to the face of the support (in.)

d = distance from the compression face to the centroid of tension reinforcement (in.)

Case 2:

A load causing more than ½ of the shear at a support is closer than $2d$ from the face of the supports.

In the past, bent caps were typically designed as "conventional beams" in accordance with the Load Factor Design (LFD) method in Caltrans Bridge Design Specifications (Caltrans, 2000), which was applicable until 2008. The LFD code for flexural design is based on the assumption that plane sections remain plane after loading and that the longitudinal strains vary linearly over the depth of the beam. Furthermore, it assumes that the shear distribution remains uniform. In bent caps, these assumptions may not always be valid. However, the sectional beam method has proved to be acceptable as it generally yields more conservative designs in regions near discontinuities. Furthermore, historical data does not suggest design inadequacies for bent caps. Caltrans will continue to use the sectional method except in very irregular beam geometries.

12.2 LOADS ON BENT CAPS

This section discusses the type of loads that bent caps must be designed to resist and support.

12.2.1 Permanent Loads

Permanent loads and forces that are, or are assumed to be, either constant or varying over a long time interval upon completion of construction.

For bent cap design, the permanent loads to consider include:

- Dead load of structural components and nonstructural attachments (DC):
  - Bridge weight: In cast-in-place box girder superstructures, normal weight concrete is 150 pcf, including the weight of bar reinforcing steel and lost formwork
  - Weight of barrier
  - Any other type of permanent attachment, such as sound walls or sign structures
- Dead load of wearing surfaces and utilities (DW), including all dead loads added to the bridge after it is constructed:
  - For new bridges, and in accordance with MTD 15-17 (Caltrans, 1988), 35 pounds per square foot must be applied on the bridge deck between the faces of barrier rails to account for three inches of future wearing surface.
  - Weight from utilities: For example, a 24-inch water line would consist of a uniformly distributed load from the pipe, hardware, and support blocks, as well as the water conveyed in the line.
- Force effects due to creep (CR): CR are a time-dependent phenomenon of concrete structures due to sustained compression load. As such, bent caps are generally not affected by the displacement-generated loads unless they are prestressed.
- Force effects due to shrinkage (SH): The SH of concrete structures are a time-dependent phenomenon that occurs as the concrete cures. The effects of shrinkage are typically not considered unless the bent cap is unusually long (wide structures). Shrinkage, like creep, also affects prestressed bent caps by creating a loss in prestress force as the structural member shortens beyond the initial elastic shortening.
- Secondary forces from post-tensioning (PS): The primary post-tensioning forces counteract dead and live load demands. However, PS

forces introduce load into the members of statically indeterminate bent caps as the cap beams shorten elastically toward the point of no movement.

- Miscellaneous locked-in force effects resulting from construction processes (EL)

Generally, DC and DW are distributed by tributary area (or width) for precast prestressed I-girder, steel girder, and bulb T girder bridges. In other types of structures, DC and DW may be distributed equally to each girder despite varying girder spacing. Those types of structures, such as cast-in-place prestressed concrete box girder sections, are so stiff that dead loads are distributed nearly equally to each girder. The self-weight of the bent cap, $DC_{bent\ cap}$, however, is distributed along the length of the bent cap as a tributary load.

12.2.2 Transient Loads

Transient Loads are loads and forces that are, or are assumed to be, varying over a short time interval. A transient load is any load that will not remain on the bridge indefinitely. For bend cap design, this includes vehicular live loads (LL) and their secondary effects including dynamic load allowance (IM), braking force (BR), and centrifugal force (CE). Additionally, there may be pedestrian live load (PL), force effects due to uniform temperature (TU), and temperature gradient (TG), force effects due to settlement (SE), water load and stream pressure (WA), wind load on structure (WS), wind on live load (WL), friction force (FR), ice load (IC), vehicular collision force (CT), vessel collision force (CV), and earthquake load (EQ).

The primary transient load that the bent cap must support is live load. Force effects from live loads are determined similarly to the methods used for the longitudinal girder analysis—through the use of an analytical process that may involve influence lines. The process of calculating wheel line loads to apply to the bent cap model involves extraction of the unfactored bent reactions for each design vehicle class from the longitudinal analysis model. Note that the reactions are generated for a single truck or lane load for each of the three vehicle classes: LRFD HL-93, Caltrans permit vehicles (P-load) (Caltrans, 2014), and fatigue vehicle.

12.2.2.1 Number of Live Load Lanes

Live load lanes (Figure 12.2-1) are not the same as the striped lanes on bridges. For bent design, force effects from a single lane of vehicular live load are acquired from the longitudinal frame analysis. To perform an analysis at the bent, various configurations of a single lane or multiple lanes are considered. Fractional lanes are not allowed for bent cap design, meaning only whole numbers of 12-ft lanes are employed.

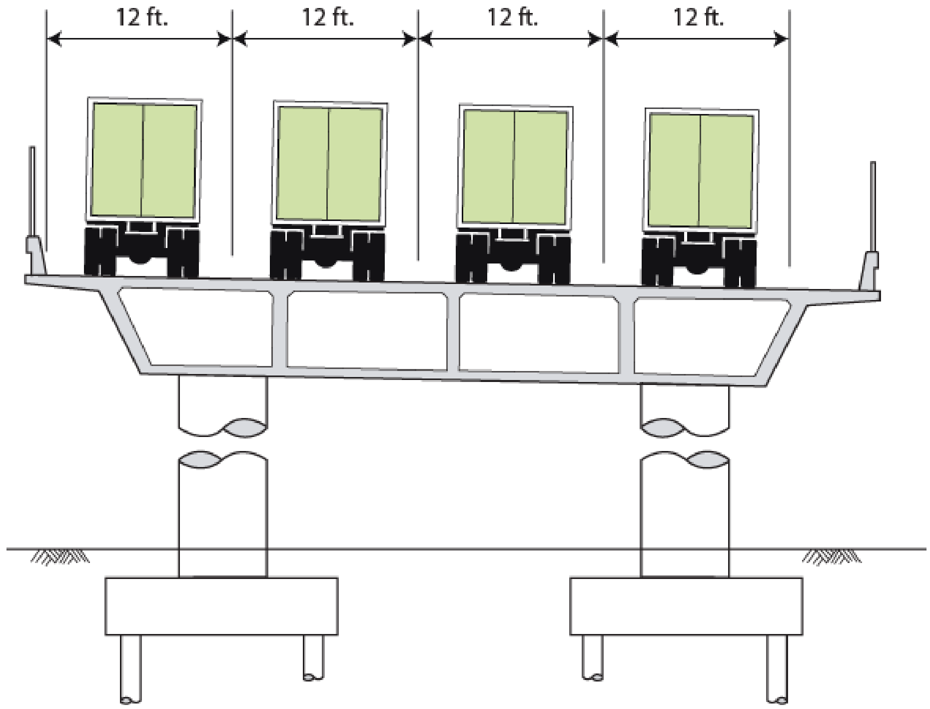


Figure 12.2-1 Number of Live Load Lanes

Maximum number of live load lanes within a bridge, N, is determined by following equation:

$$N = \text{Integer part of } \{[\text{width of bridge (ft)} - \text{barrier widths (ft)}] / 12 \text{ ft}\} \qquad (12.2\text{-}1)$$

Per AASHTO Article 3.6.1.1.1, future changes to clear roadway width should be considered. The lane load is considered uniformly distributed over a 10-ft width. However, designers may simplify the analysis by combining the HL-93 lane load with the HL-93 truck wheel line load. Note that, per AASHTO 3.6.1.2.4, the dynamic load allowance, IM, shall be applied only to the truck load.

12.2.2.2 Multiple Presence Factors, *m*

Multiple presence factors, m, as specified in AASHTO Table 3.6.1.1.2-1, are used to account for the improbability of fully loaded trucks crossing the structure simultaneously and are applied to the vehicular live loading.

12.2.2.3 Vehicular Live Load Positioning

An important consideration in the design of bent caps is to determine the maximum or critical force effects by positioning live load lanes. The location of the truck and lane, as shown in Figure 12.2-1, has an important bearing on the force effect on the bent cap. Whether the truck is at midspan of a bent cap or at the support, it has an effect on the value of the moment and shear on the bent cap.

AASHTO LRFD requires that one truck be placed in each lane transversely: If the bridge can fit four lanes, then up to four trucks can be placed on the bridge, one in each lane.

Lanes are placed to produce the maximum force effects in the bent cap. However, one must consider the effect of the multiple presence factor, *m*, as it is not always evident that placing the maximum number of trucks will garner the maximum force effects in the bent cap. For example, for closely spaced columns within a bent cap, two HL-93 lanes may result in greater shear demand than four HL-93 lanes because the latter case would require a multiple presence factor of 0.65.

The designer must also consider that a certain configuration, and number, of live load lane positions may result in maximum shear effects but not necessarily maximum moment effects. For a bent cap supported by multiple columns, it is advisable to use a structural analysis program that is capable of generating combinations of lane configurations, as well as influence lines from moving live loads. CsiBridge, and CTBridge are such programs.

12.2.2.4 HL-93 Design Vehicular Live Load Positioned Transversely

HL-93 consists of design truck, or design tandem, and design lane load. Figure 12.2-2 shows one of two alternatives for a design truck, or wheel lines, transversely placed within a 12-ft live load lane. The other alternative is a mirror image of this graphic depiction. The wheel lines may move anywhere within the 12-ft lane as long as AASHTO 3.6.1.3.1 is satisfied. Lanes and wheel lines shall be placed to produce maximum force effects in the bent cap.

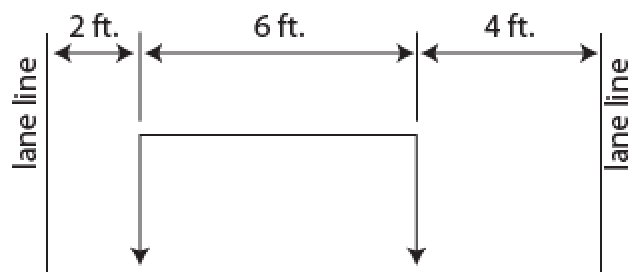


Figure 12.2-2 HL-93 Design Truck Positioned Transversely

When multiple lanes are applied to the bent cap, the wheel lines may be positioned as shown in Figure 12.2-3:

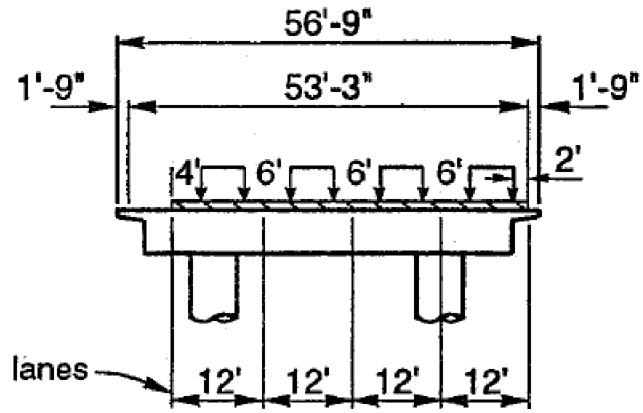


Figure 12.2-3 Wheel Line Spacing for Four HL-93 Trucks

Figure 12.2-4 shows a 10-ft wide HL-93 lane load placed in 12-ft wide lanes.

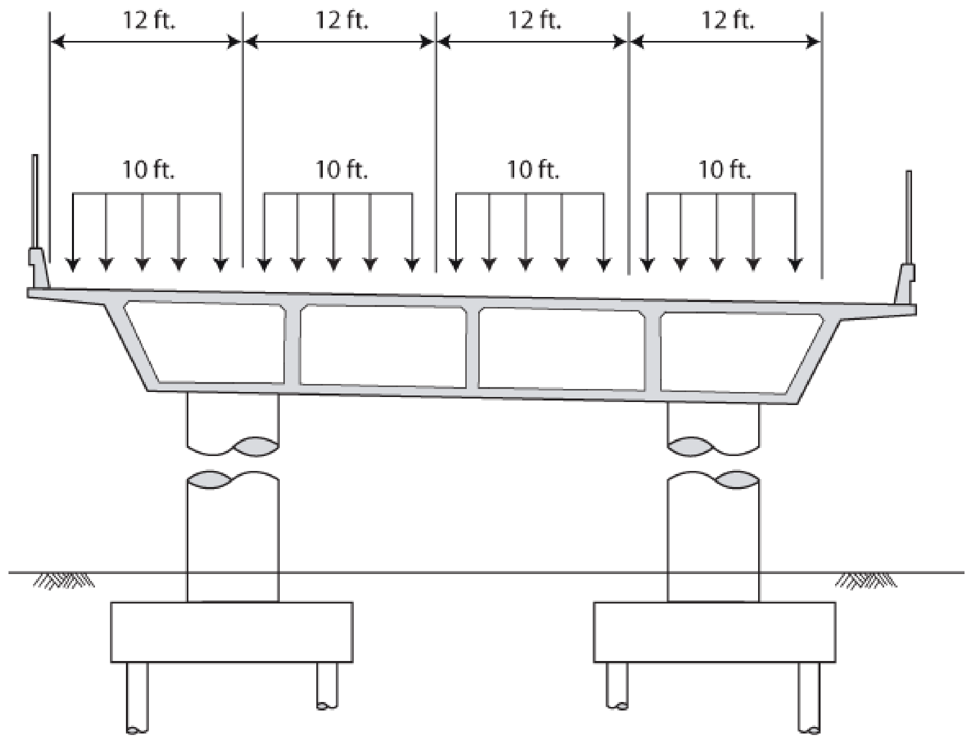


Figure 12.2-4 HL-93 Lane Loads Positioned Transversely

12.2.2.5 Permit Trucks Positioned Transversely

Per CA 3.4.1 (Caltrans 2014), for bent cap design, a maximum of two permit trucks shall be placed in lanes that are positioned to create the most severe condition. Figure 12.2-5 shows two permit trucks occupying two adjacent lanes. However, the lanes may be positioned apart if that results in maximum bent cap force effects.

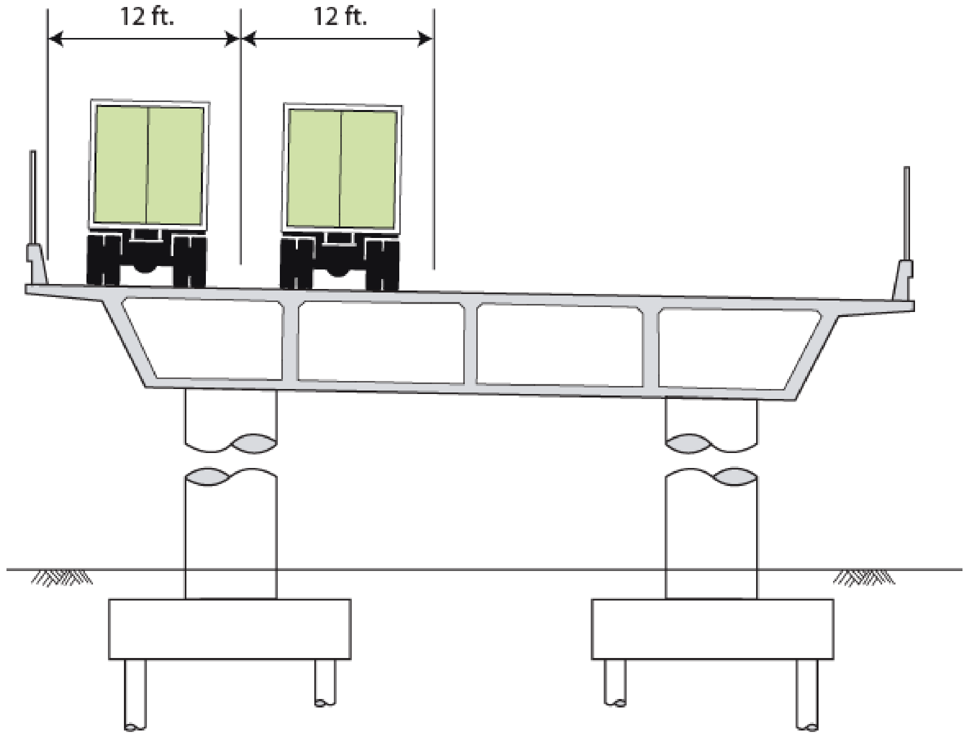


Figure 12.2-5 Permit Trucks Positioned Transversely

12.3 DESIGN CONSIDERATIONS

This section presents the typical design procedure for cast-in-place concrete bent caps. Topics include design for flexure, shear, and shear-flexure interaction.

12.3.1 Flexural Design

Reinforced concrete bent caps shall be designed to satisfy the strength, service, and fatigue limit states.

The goal of flexural design under the strength limit state is to provide enough resistance to satisfy the strength limit state conditions. This may be achieved by using bar reinforcing steel or prestressing in the cast-in-place concrete bent cap.

12.3.1.1 Flexural Design Process

The flexural design process for bent caps consists of 12 primary steps, summarized below:

1) Calculate factored moments for Strength I and II limit states
2) Calculate minimum cracking moment, then determine minimum design moment, M_{min}
3) Determine the factored moment demand, M_u
4) Assume an initial value for area of nonprestressed tension reinforcement, A_s
5) Calculate net tensile strain, ε_t, and determine resistance factor, ϕ
6) Determine whether the section is rectangular or flanged
7) Calculate the average stress in prestressing steel, f_{ps}, if the bent cap is post-tensioned
8) Calculate the nominal flexural resistance, M_n
9) Calculate the factored flexural resistance, M_r
10) Iterate steps 4 through 9 until $M_r \geq M_u$ and the design assumptions are verified
11) Check for serviceability
12) Check for fatigue

12.3.1.1.1 Determine the Factored Moment from Strength I and Strength II Limit States

Per CA table 3.4.1−1, both Strength I and Strength II limit states are used to calculate M_u for bent caps. For simplicity, Strength III through V, are not being considered as Strength I and II will govern in most bent cap designs. For additional simplicity, moment demand from prestressing, creep, shrinkage, stream pressure, uniform temperature change, temperature gradient, and settlement are not being considered:

- Strength I

$$M_{u(HL93)} = 1.25(M_{DC}) + 1.5(M_{DW}) + 1.75(M_{HL93}) \qquad (12.3.1\text{-}1)$$

- Strength II

$$M_{u(\mathrm{P-15})} = 1.25(M_{DC}) + 1.5(M_{DW}) + 1.35(M_{P-15}) \qquad (12.3.1\text{-}2)$$

where:

$M_{u(HL93)}$ = factored moment demand at the section from HL93 Vehicle

$M_{u(\mathrm{P-15})}$ = factored moment demand at the section from the Permit Vehicle

M_{DC} = unfactored moment demand at the section from dead load of structural components and nonstructural attachments

M_{DW} = unfactored moment demand at the section from dead load of wearing surfaces and utilities

From the above two limit states, the larger of the two values is the controlling moment. It is possible to have different limit states control at different locations along the bent cap.

12.3.1.1.2 Calculate Minimum Reinforcement Design Moment, $M_{\min}$

The minimum reinforcement requirement ensures that the flexural design of the bent cap provides either enough post-cracking ductility of the member or a modest margin of safety over M_u:

$$M_{cr} = \gamma_3\left[(\gamma_1 f_r + \gamma_2 f_{cpe})S_c - M_{dhc}\left(\frac{S_c}{S_{nc}} - 1\right)\right] \qquad (\text{AASHTO } 5.7.3.3.2\text{−}1)$$

$$S_c = \frac{I_g}{Y_t} \qquad (12.3.1\text{-}3)$$

where:

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

I_g = moment of inertial of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4})

Y_t = distance from the neutral axis to the extreme tension fiber (in.)

f_r = modulus of rupture of the concrete (ksi)

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) not including the effects of secondary moment, at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

γ_1 = flexural cracking variability factor

γ_2 = prestress variability factor

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement

M_{cr} = cracking moment (kip-in.)

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3})

M_{dnc} and S_{nc} typically apply to precast girders before and after the girder become composite with the deck. Therefore, the terms do not typically apply to cast-in-place bent cap design and will be disregarded. Since the design examples are of non-prestressed bent caps, remove f_{cpe} from the equation, and reduce the formula to:

$$M_{cr} = \gamma_3 (\gamma_1 f_r) S_c \qquad (12.3.1\text{-}4)$$

Per AASHTO 5.7.3.3.2, the minimum factored moment demand is:

$$M_{min} = \min (M_{cr}, 1.33M_u) \qquad (12.3.1\text{-}5)$$

12.3.1.1.3 *Determine the Factored Moment Demand, M_u*

The maximum value of M_u is:

$$M_u = \max(M_{u(HL93)}, M_{u(P\text{-}15)}, M_{min}) \qquad (12.3.1\text{-}6)$$

12.3.1.1.4 *Determine the Resistance Factor, ϕ*

The AASHTO code requires that factored loads be less than or equal to factored resistances, as shown below:

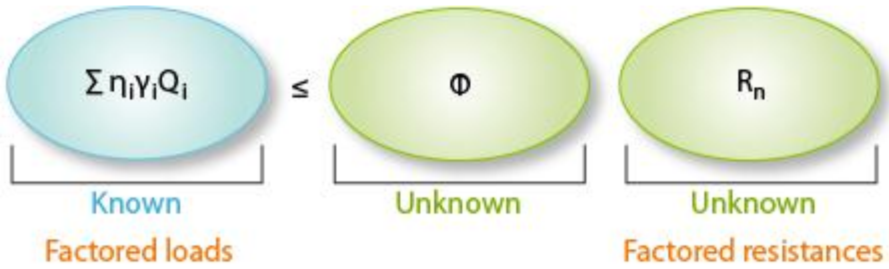


Figure 12.3-1 General Equation of LRFD Methodology

For flexure design, the AASHTO LRFD code specifies a variable resistance factor. The relationship between ϕ and the steel net tensile strain, ε_t, is provided in the design specifications. To determine the resistance factor for flexure design of the bent cap, the designer must first calculate the steel net tensile strain.

12.3.1.1.5 *Calculate the Section's Net Tensile Strain, ε_t*

The net tensile strain, ε_t, is the tensile strain in the extreme tension steel at nominal flexural strength.

The nominal flexural strength is reached when the concrete strain in the extreme compression fiber reaches the assumed ultimate strain of 0.003. The following graphic shows a rectangle section with linear strain distribution along the section.

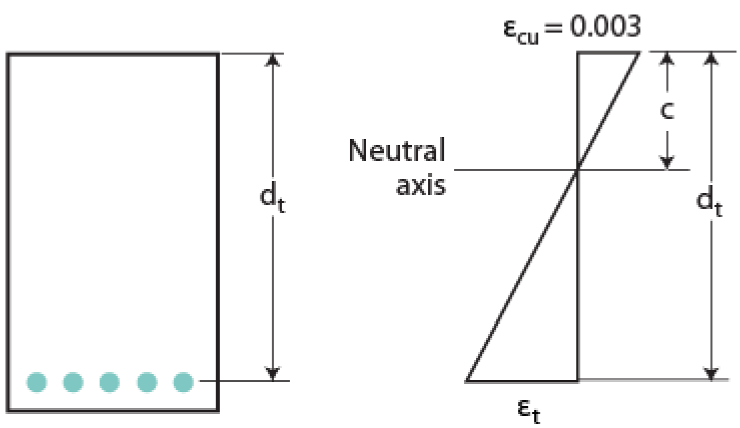


Figure 12.3-2 Strain Diagram for Rectangular Section

where:

c = distance from the extreme compression fiber to the neutral axis

d_t = distance from the extreme compression fiber to the centroid of extreme tension steel

ε_{cu} = failure strain of concrete in compression

Based on AASHTO 5.7.2.1, the strain along the section is a linear distribution or directly proportional to the distance from the neutral axis. Using the similar triangle method, the steel net tensile strain can be determined as:

$$\varepsilon_t = \varepsilon_{cu}\left(\frac{d_t}{c-1}\right) = 0.003\left(\frac{d_t}{c-1}\right) \tag{12.3.1-7}$$

The following graph from CA C5.5.4.2.1-1 shows how the resistance factor, ϕ, varies with the section's net tensile strain ε_t.

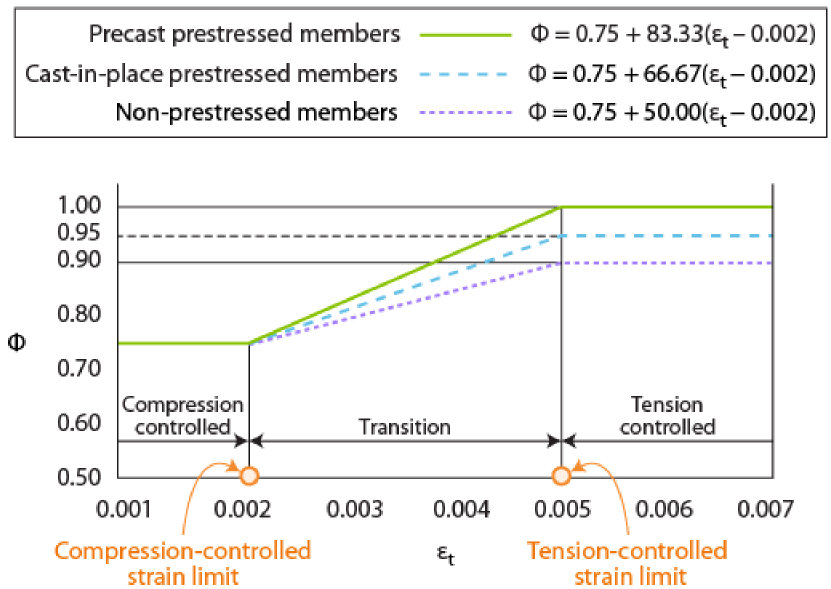


Figure 12.3-3 Variation of ϕ with Net Tensile Strain, ε_t

Note that the variation of ϕ has three distinct linear regions. These regions describe ranges of ε_t that denote whether a section is compression-controlled, tension-controlled, or a transition section.

- Compression-controlled section

  If $\varepsilon_t \leq 0.002$, the section is defined as a compression-controlled section. Under external load action, a compression-controlled member fails in a brittle manner with little warning. To avoid this scenario, the design code requires a more conservative resistance factor, $\phi = 0.75$.

- Tension-controlled section

  If $\varepsilon_t \geq 0.005$, the section is defined as a tension-controlled section. Failure of a tension-controlled member is more ductile and is considered to have sufficient warning before failure by means of deflection and cracking. Therefore, the design specifications specifies a relatively large resistance factor for tension-controlled members.

- Transition section

  If ε_t is located between the above two limits ($0.002 < \varepsilon_t < 0.005$), the section is defined as a transition section. Under this condition, the resistance factor will vary linearly with ε_t.

The following figure illustrates the net tensile strain limits:

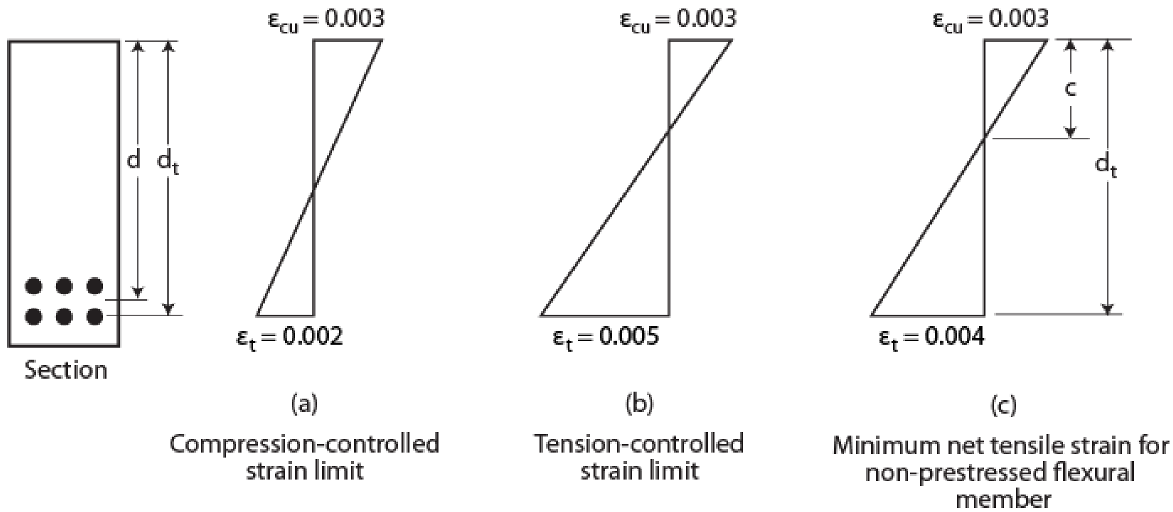


Figure 12.3-4 Net Tensile Strain Limits.

Note: Per CA C5.7.2.1, the net tensile strain for a reinforced (non-prestressed) concrete flexural member, such as a bent cap, shall not be less than 0.004.

12.3.1.1.6 Determine the Factored Flexural Resistance, M_r

The factored flexural resistance is:

$$M_r = \phi M_n \tag{12.3.1-8}$$

12.3.1.1.7 Determine whether the Section is Rectangular or Flanged

For monolithic integral bent caps, it is Caltrans' practice to consider part of the deck and soffit slab to be working with the solid bent cap section as a flanged cross-section as shown in Figure 12.3-5.

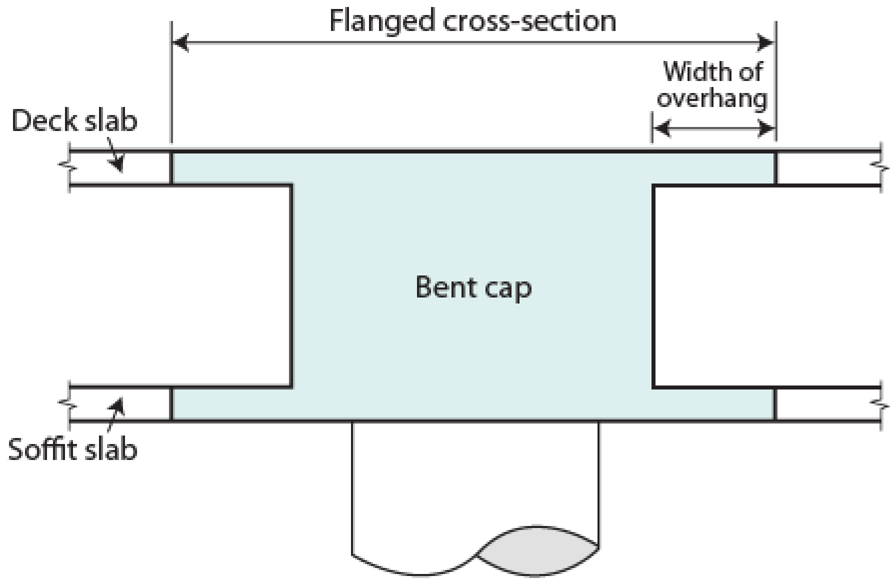


Figure 12.3-5 Flanged Cross Section

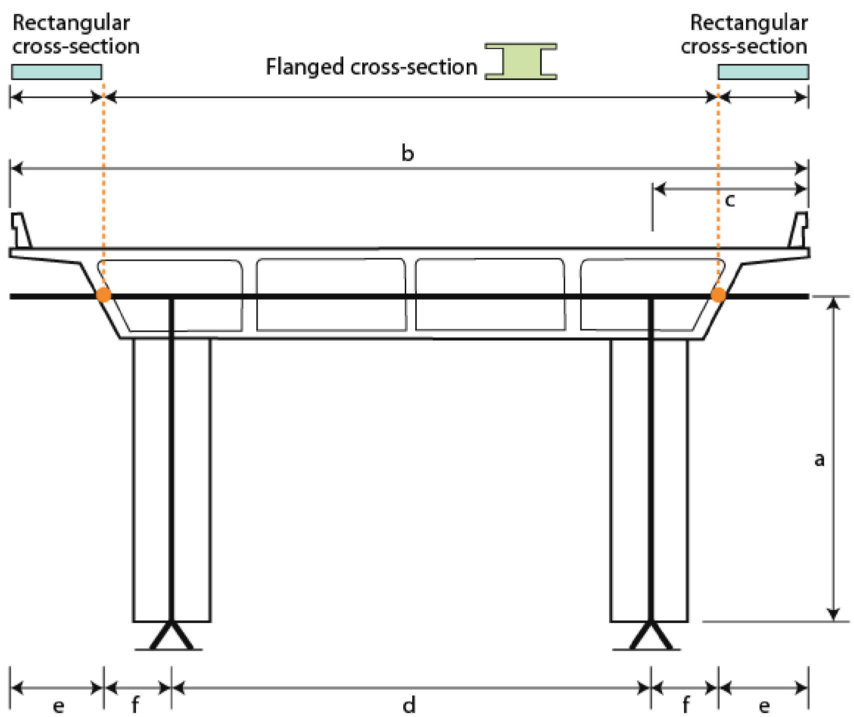


Figure 12.3-6 Geometric Model

where :

a = distance from the center of gravity (CG) of the superstructure to the bottom of column = height of the column + depth to CG of the bent cap

b = top width of superstructure

c = distance from the centerline (CL) of the columns to the edge of deck ($e + f$)

d = distance between the CL of columns

e = distance from the CL of exterior girder to the edge of deck (EOD)

f = distance from the CL of the column to the CL of the exterior girder

In the geometrical model shown in Figure 12.3-6, the flanged cross-section spans from the centerline of the left exterior girder to the centerline of the right exterior girder at the CG of the bent. The rectangular cross-sections span from the centerline of the exterior girder to the edge of deck. These sections are assumed as rectangular in order to simplify the analytical model.

For drop bent caps, the section is rectangular for the full length of the bent cap.

Although the terms "flanged section" and "rectangular section" describe the geometric section, it is important to note that they are not necessarily accurate depictions of the analytical section. A "flanged section" may be analyzed as a "rectangular section." The scenario exists when the depth of the compression zone, i.e, distance from the extreme compression fiber to the neutral axis, c, is less than the thickness of the compression flange, h_f ($c \leq h_f$). If the depth of the compression zone is greater than the thickness of the compression flange ($c > h_f$), then the section will exhibit "flanged section" behavior.

If the section is a rectangular section, then determine c as follows:

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A_s^{'} f_s^{'}}{0.85 f_c^{'} \beta_1 b + k A_{ps} \left(\frac{f_{pu}}{d_p} \right)} \qquad \text{(AASHTO 5.7.3.1.1-4)}$$

If the section is a flanged section, then determine c as follows:

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A_s^{'} f_s^{'} - 0.58 f_c^{'} (b - b_w) h_f}{0.85 f_c^{'} \beta_1 b_w + k A_{ps} \left(\frac{f_{pu}}{d_p} \right)} \qquad \text{(AASHTO 5.7.3.1.1-3)}$$

where:

A_{ps} = area of prestressing steel (in.^{2})

f_{pu} = specified tensile strength of prestressing steel (ksi)

c = distance from extreme compressive fiber to the neutral axis (in.)

d_p = distance from extreme compression fiber to the centroid of prestressing tendons (in.)

A_s = area of mild steel tension reinforcement (in.^{2})

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi)

d_s = distance from extreme compression fiber to the centroid of non−prestressed tensile reinforcement (in.)

A'_s = area of compression reinforcement (in.^{2})

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi)

d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.)

f'_c = specified compressive strength of concrete (ksi) at 28 days, unless another age is specified (ksi)

b = width of the compression face of the member (in.)

b_w = web width or diameter of a circular section (in.)

h_f = compression flange depth of an I or T member (in.)

β_1 = stress block factor specified in AASHTO Article 5.7.2.2

For $f'_c \leq 4$ ksi, $\beta_1 = 0.85$

For $f'_c \geq 4$ ksi, β_1 is reduced at a rate of 0.05 for each 1 ksi of strength in excess of 4 ksi, except $\beta_1 \geq 0.65$

f_{ps} = average stress in prestressing steel; value is often zero for non-prestressed bent caps

The average stress in prestressing steel, f_{ps}, may be taken as:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{(AASHTO 5.7.3.1.1−1)}$$

k = constant that depends on the type of tendon used:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad \text{(AASHTO 5.7.3.1.1-2)}$$

Alternatively, the following table can be used:

AASHTO Table C5.7.3.1.1-1 Value of *k*

Type of Tendon	$\frac{f_{py}}{f_{pu}}$	Value of k
Low relaxation strand	0.9	0.28
Stress-relieved strand	0.85	0.38
Type 1 high-strength bar	0.85	0.38
Type 2 high-strength bar	0.8	0.48

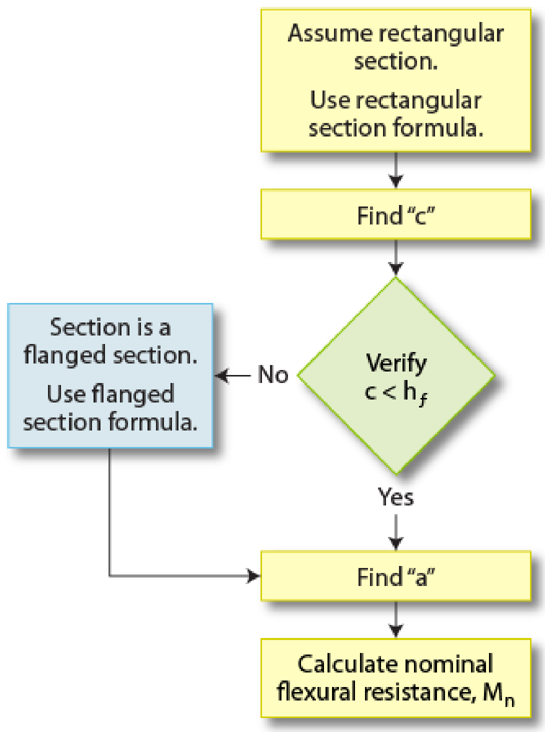


Figure 12.3-7 Flexural Resistance Calculation Flowchart

M_n is the nominal resistance, which can be calculated by:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + 0.85 f'_c\left(b - b_w\right) h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

(AASHTO 5.7.3.2.2-1)

where:

A_{ps} = area of prestressing steel (in.^{2})

f_{ps} = average stress in prestressing steel at nominal bending resistance (ksi)

d_p = distance from extreme compression fiber to the centroid of prestressing tendons (in.)

A_s = area of mild steel tension reinforcement (in.^{2})

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi)

d_s = distance from extreme compression fiber to the centroid of non−prestressed tensile reinforcement (in.)

A'_s = area of compression reinforcement (in.^{2})

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi)

d'_s = resistance from extreme compression fiber to the centroid of compression reinforcement (in.)

f'_c = specified compressive strength of concrete (ksi) at 28 days, unless another age is specified (ksi)

b = width of the compression face of the member (in.)

b_w = web width or diameter of a circular section (in.)

h_f = compression flange depth of an I or T member (in.)

a = $c\beta_1$; depth of the equivalent stress block (in.)

β_1 = stress block factor specified in AASHTO Article 5.7.2.2

The graphic below shows the variables that comprise each component of the formula:

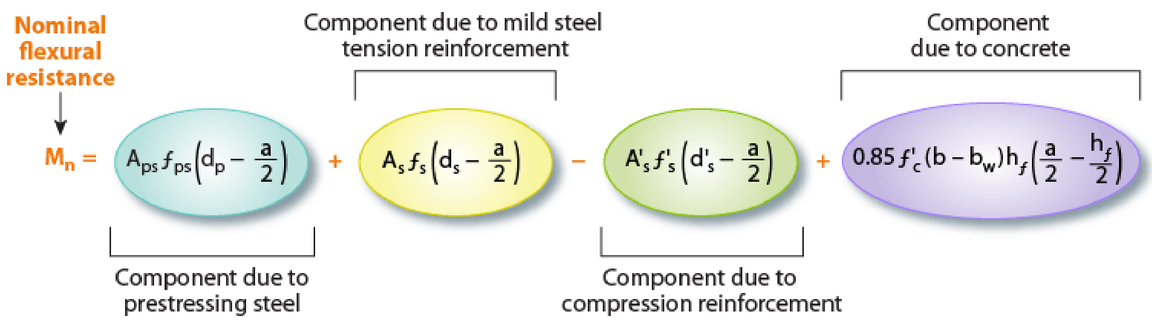


Figure 12.3-8 Components of Flexural Moment

For non-prestressed cast-in-place bent caps, in which compression reinforcement is ignored, the term for M_n can be reduced to:

$$M_n = A_s f_s \left(d_s - \frac{a}{2} \right) + 0.85 f_c' (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2} \right) \tag{12.3.1-9}$$

12.3.1.2 Check for Serviceability

Cracks occur in concrete components due to:

- Loading conditions
- Thermal effects
- Deformations

Cracks occur whenever tension stress in the member exceeds the modulus of rupture of concrete. The severity of flexural cracking in a concrete bent caps can be controlled by providing optimized tension reinforcement layouts, limiting bar sizes, and providing tighter spacing.

Per AASHTO 5.7.3.4, the spacing, *s*, of mild steel reinforcement in the layer closest to the tension face is given by:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \tag{AASHTO 5.7.3.4-1}$$

where:

d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest flexural reinforcement (in.)

γ_e = exposure factor

= 1.00 for Class 1 exposure condition

= 0.75 for Class 2 exposure condition

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} \tag{12.3.1-10}$$

h = overall thickness or depth of the component (in.)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)

Note: In the above equation, the spacing, *s*, of the bar reinforcing steel is inversely proportional to the stress in the reinforcing steel.

Also, per CA 5.7.3.4:

- Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns for appearance and/or corrosion.
- Class 2 exposure condition applies when there are increased concerns for appearance and/or corrosion (for example; in areas where de-icing salts are used). Class 2 exposure condition applies to all bridge deck.

12.3.1.3 Check for Fatigue

As per AASHTO 5.5.3.1, the stress range in reinforcing bars due to the fatigue load combination shall satisfy:

$$\gamma(\Delta f) \leq (\Delta F)_{TH} \quad \text{(AASHTO 5.5.3.1-1)}$$

$$(\Delta F)_{TH} = 24 - 0.33 f_{min} \quad \text{(AASHTO 5.5.3.2-1)}$$

where:

γ = load factor for Fatigue I

Δf = live load stress range (ksi)

f_{min} = minimum live load stress resulting from the Fatigue I load combination, combined with more serve stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; . positive if tension, negative if compression (ksi)

For the fatigue check:

- The fatigue load combination is given in CA Table 3.4.1-1 and features a load factor of 1.75 for the infinite fatigue life.
- Apply the IM factor to the fatigue load.
- Check both top and bottom reinforcement to ensure that the stress range in the reinforcement under the fatigue load stays within the range specified in the above equation.

12.3.2 Design for Shear

The shear design of bent caps involves:

- Determining the stirrup bar size along the length of the bent cap
- Determining stirrup spacing along the bent cap
- Checking shear-flexure interaction

The LRFD shear design method is based on the Modified Compression Field Theory. Contrary to the traditional shear design methodology, it assumes a variable angle truss model instead of the 45º truss analogy. The LRFD method also accounts for interaction between shear, torsion, flexure, and axial load, as well as residual tension in concrete after cracking, which was neglected in the traditional method of shear design. California Amendments Article 5.8.3.4.2 specifies that shear resistance of all prestressed and nonprestressed sections shall be determined by AASHTO Appendix B5.

The LRFD design code notes that shear design may be considered to the distance, d_v, from the face of support (AASHTO 5.8.3.2). However, Caltrans' practice is to evaluate shear to the face of support. See Chapter 5 for detailed discussion.

Factored shear resistance, V_r, is given by:

$V_r = \phi V_n$ (AASHTO 5.8.2.1−2)

The nominal shear resistance, V_n, is the lesser of:

$V_n = V_c + V_s + V_p$ (AASHTO 5.8.3.3−1)

or

$V_n = 0.25 f'_c b_v d_v + V_p$ (AASHTO 5.8.3.3−2)

If if the procedures of AASHTO 5.8.3.4.1 or 5.8.3.4.2 are used

$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v$ (AAAHTO 5.8.3.3-4)

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$$ (AASHTO 5.8.3.3−4)

f'_c = specified compressive strength of concrete (ksi)

f_y = yield strength of transverse reinforcement (ksi)

V_p = component in the direction of the applied shear of the effective prestressing force (kip); positive if resisting the applied shear; typically zero for conventionally reinforced bent caps

b_v = effective web width (in.) taken as the minimum web width within the depth d_v

d_v = effective shear depth (in.)

s = spacing of stirrups (in.)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear

θ = angle of inclination (°) of diagonal compressive stresses

α = angle of inclination (°) of transverse reinforcement to longitudinal axis; typically 90°

A_v = area of shear reinforcement (in.^{2}) within a distance s

The main shear equation may be rearranged and simplified for design purposes, as such:

$$V_s \geq \frac{V_u}{\phi} - V_c$$ (12.3.2-1)

Per AASHTO C5.8.2.4, transverse reinforcement must be provided in all regions where there is a significant chance of diagonal cracking. Transverse reinforcement must be provided where:

$V_u > 0.5\phi (V_c + V_p)$ (AASHTO 5.8.2.4−1)

Determine the effective shear depth, d.

As specified in AASHTO 5.8.2.9, the effective shear depth, d_v, is taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure.

The effective shear depth, d_v, is given by:

$$d_v = d_e - \frac{a}{2} \tag{12.3.2-2}$$

where:

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

d_v needs not be taken to be less than the greater of $0.9d_e$ or $0.72h$ where h is the overall thickness or depth of a member

12.3.2.1 Determine Factor β and Angle θ

Using shear stress ratio, v_u / f'_c, and longitudinal strain, ε_x, to find values of β and θ from AASHTO Table B5.2-1.

For non-prestressed concrete bent cap,

$$v_u = \frac{V_u}{\phi b_v d_v} \qquad \text{(AASHTO 5.8.2.9-1)}$$

where:

V_u = factored shear (kip)

b_v = effective web width taken as the minimum web width (in.)

d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compression force due to flexure, it need not be taken to be less than the greater of $0.9d_e$ or $0.72h$ (in.)

The longitudinal strain at the middepth of member ε_x, shall be determined by:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps} f_{po}\right)}{2\left(E_s A_s + E_p A_{ps}\right)} \qquad \text{(AASHTO B5.2-1)}$$

where:

$|M_u|$ = absolute value of the factored moment, not to be taken less than $|V_u - V_p| d_v$ (kip-in.)

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

V_u = factored shear force (kip)

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and surrounding concrete (ksi)

E_s = modulus of elasticity of reinforcing bars (ksi)

E_p = modulus of elasticity of prestressing tendons (ksi)

Note: The longitudinal strain is a function of the desired angle of compression strut, θ. This necessitates an iterative procedure to solve for the longitudinal strain, ε_x, and θ. Thus, it may be useful to assume that the term (0.5cotθ) equals 1.0 and reduce the number of iterations.

12.3.2.2 Determine the Amount of Shear Stirrups, A_v, and Spacing, s

Knowing, V_u, V_c, and ϕ, the demand on shear stirrups, V_s, can be determined:

$$V_s \geq \frac{V_u}{\phi} - V_c \tag{12.3.2-1}$$

Knowing V_c, the term $\frac{A_v}{s}$ can be determined:

$$\frac{A_v}{s} = \frac{V_s}{f_y d_v (\cot\theta + \cot\alpha)\sin\alpha} \tag{12.3.2-2}$$

Check for minimum shear reinforcement:

$$\left(\frac{A_v}{s}\right)_{mim} = 0.0316\frac{\sqrt{f'_c}\, b_v}{f_y} \qquad \text{(AASHTO 5.8.2.5-1)}$$

Check for maximum spacing:

For $v_u / f'_c < 0.125$, $s_{max} = 0.8d_v \leq 18$ in. (CA 5.8.2.7-1)

For $v_u / f'_c \geq 0.125$, $s_{max} = 0.4d_v \leq 12$ in. (AASHTO 5.8.2-7-2)

Repeat for other points along bent cap span (typically at tenth point) and obtain A_v and s.

12.3.3 Check Longitudinal Steel for Tension (Shear-Flexure Interaction)

The effect of shear forces on the longitudinal reinforcement is determined and the adequacy of the reinforcement is checked using the AASHTO interaction equation 5.8.3.5-1. Longitudinal reinforcement along with the vertical steel stirrups and the compression strut in concrete constitute the truss mechanism that carries the applied loads.

The check of the adequacy of the longitudinal reinforcement may result in added length of the longitudinal bar reinforcing and/or added amount of longitudinal reinforcement. The former occurs if the original bar reinforcement is curtailed, and the latter occurs in case of indirect loading/support such as the case of box girders framing into bent caps. Figure 12.3-9 shows the concept of direct/indirect loading and support, along with the demand on the longitudinal steel from flexure (solid line) and shear (dashed line).

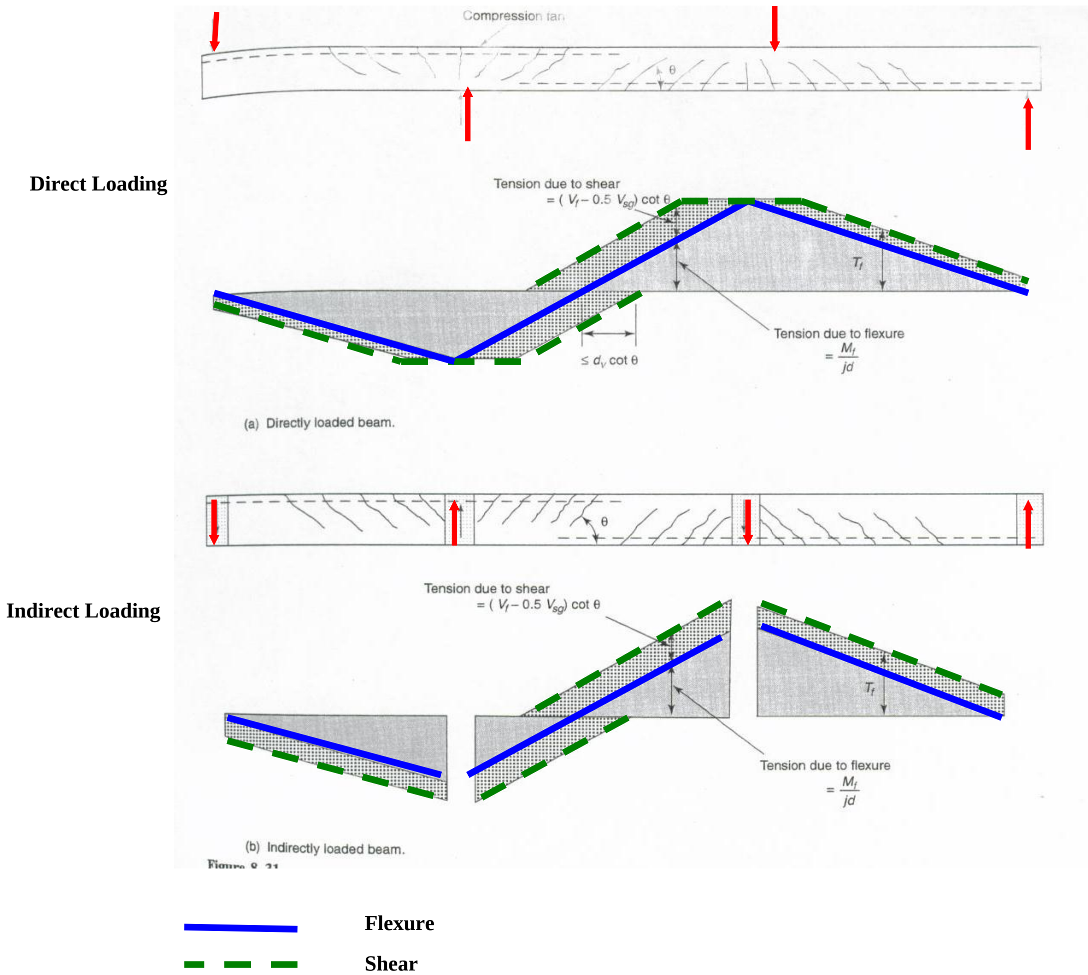


Figure 12.3-9 Direct Versus Indirect Loading/Support

As can be seen from Figure 12.3-9, the amount of longitudinal steel need not exceed the maximum amount due to flexure demands in the case of direct loading/support. The additional shear demands on the longitudinal steel can be overcome by extending the length of the longitudinal bar reinforcement. However, in the case of girders framing onto other girders at equal depth or height, the shear demand is likely to result in an additional amount of longitudinal steel beyond what is needed to meet the maximum flexure demands.

A direct loading/support case is typical for drop bent cap beams. Precast and steel girders are applying the load atop the bent cap. Columns are also directly supporting the cap. Since Caltrans practices no curtailment of the longitudinal reinforcement (due to nature of seismic loading), there is no need to check for shear-flexure interaction. Indirect loadings/supports are often encountered in box girder construction. At location of box girders framing onto the cap, the amount of bent cap longitudinal reinforcement needs to be checked using the shear-flexure interaction equation. Similar to drop bent caps, the columns are directly supporting the cap and no added longitudinal reinforcement is required at the face of column support.

The tension capacity of the longitudinal reinforcement is determined on the flexural tension side using fully developed steel and at locations of applied concentrated loads (on integral bent caps).

At every location, the following three possible load conditions shall be checked:

- Maximum shear and associated moments
- Maximum positive moments and associated shear
- Maximum negative moments and associated shear

The longitudinal steel must satisfy:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$

(AASHTO 5.8.3.5-1)

For the typical case of no prestressing and axial force in bent cap, the above equation reduces to:

$$A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + \left(\left|\frac{V_u}{\phi_v}\right| - 0.5V_s\right)\cot\theta \qquad (12.3.3\text{-}1)$$

where:

ϕ_f = resistance factor for moment

ϕ_c = resistance factor for axial load

ϕ_v = resistance factor for shear

Note: V_s shall not be taken more than $\frac{V_u}{\phi}$.

12.3.4 Design for Seismic

For seismic design, resistance factor shall be taken as $\phi = 1.0$ (CA 5.5.5).

12.4 DETAILING CONSIDERATIONS

In addition to the main top and bottom longitudinal steel for flexure and vertical stirrups for shear, reinforcement is required for side-face and end reinforcement for crack control, as well as for construction purposes.

12.4.1 Construction Reinforcement

This reinforcement is only needed for box girder construction. Concrete for the box girder is usually placed in two stages. The first stage includes placing the soffit slab and the girder stems. It is commonly known as the "stem and soffit" pour since the deck slab is not included. The second stage consists of constructing the top three to four inches of the stem and the deck slab as shown in Figure 12.4-1.

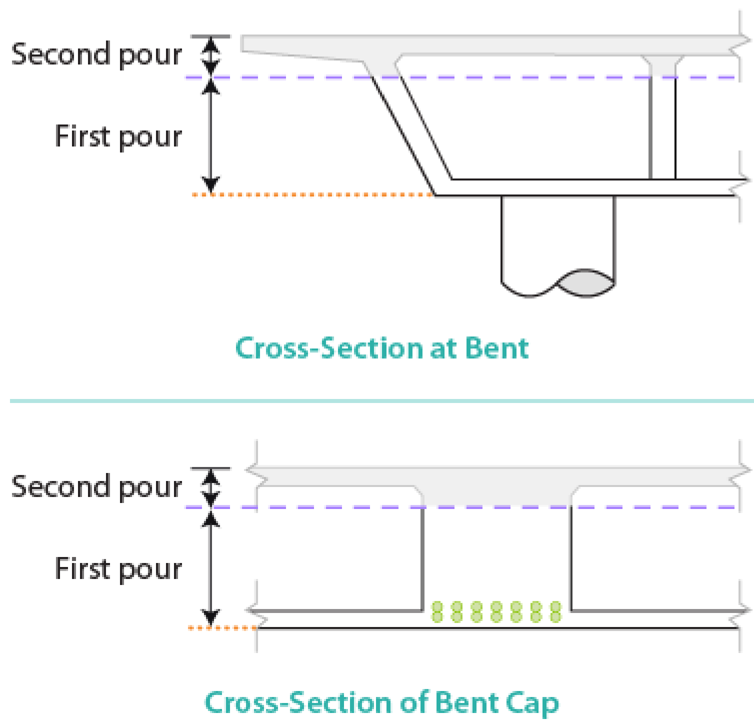


Figure 12.4-1 Concrete Pour Stage

The two-stage pouring of concrete results in a construction joint at the bent cap at the top of the girder stem. At this stage (after first pour), the girders are not stressed, longitudinal bottom steel is in place, and the entire bridge is supported on falsework.

If falsework underwent any settlement or failed unexpectedly due to impact by an errant vehicle, the bent cap will be subjected to a bending moment similar to that shown in Figure 12.4-2. As shown, there is negative moment at and near the supports (columns) with no top steel to resist this moment.

As such, it is Caltrans practice to provide additional top longitudinal steel, also commonly referred to as construction reinforcement, underneath the construction joint for the potential loss or settlement of the falsework supporting the bridge. A 10-ft length of the exterior stem girder on each side of the bent cap is assumed to act as dead load weight in addition to the bent cap own weight. Detailed weight calculations and design consideration for construction reinforcement are shown in Section 12.5.1, Integral Bent Cap Example. See Figure 12.4-3.

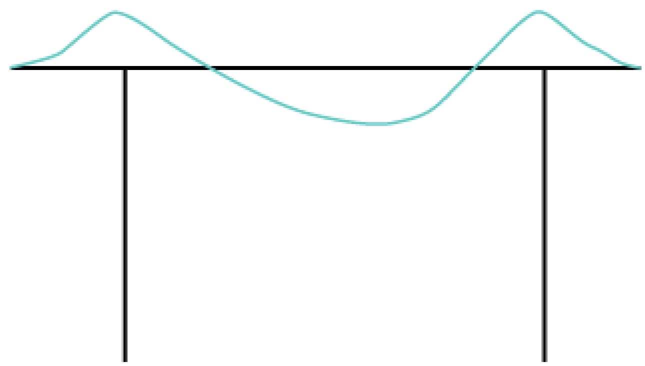


Figure 12.4-2 Potential Moment Diagram after First Pour

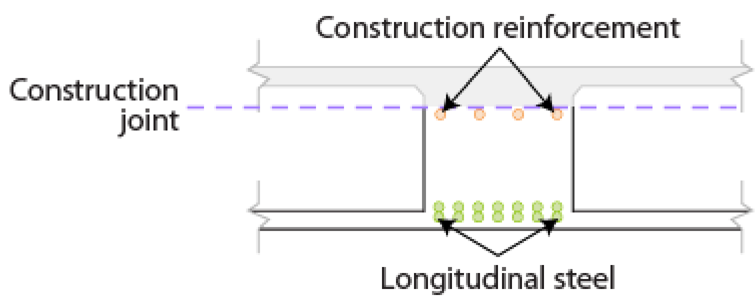


Figure 12.4-3 Construction Reinforcement

12.4.2 Side Face Reinforcement

Caltrans SDC 7.4.4.3 (Caltrans, 2013) requires side face reinforcement in quantity that amounts to 10 percent of the maximum amount of longitudinal reinforcement of the bent cap. The maximum amount of longitudinal reinforcement will come from either the top or bottom steel. The side face reinforcement, shown in Figure 12.4-4, is placed along the two vertical faces of the bent cap and shall have a maximum spacing of 12 inches.

It is permissible to include the construction reinforcement to satisfy part of this requirement.

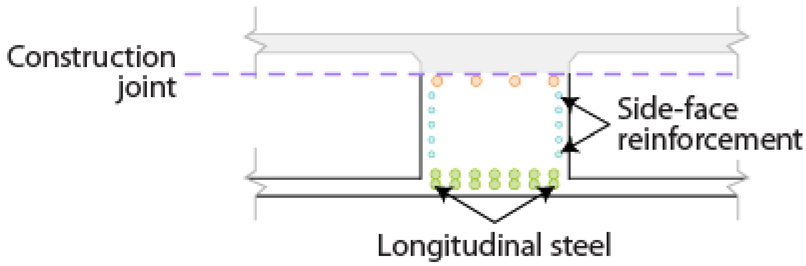


Figure 12.4-4 Side-Face Reinforcement

12.4.3 End Reinforcement

The end reinforcement is provided along the end face of the bent cap, as shown in Figure 12.4-5, as a crack control measure. There are two basic types of end reinforcement, Z-bars and U-bars as shown in Figure 12.4-5.

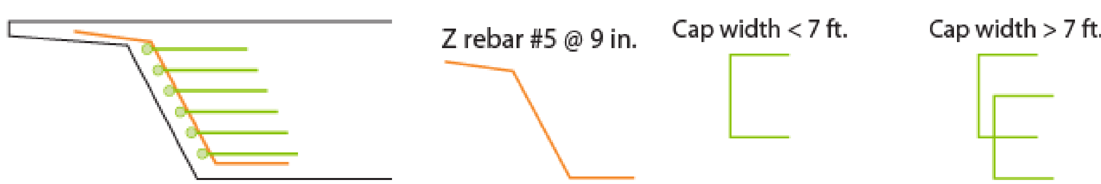


Figure 12.4-5 End Reinforcement in the Bent Cap

The U-bars (horizontal plane) are typically designed using the shear friction concept to resist the dead and live load of the exterior girder. The number of U-bars per set depends on whether the bent cap width is less than or more than seven feett.

12.4.4 Other Detailing Considerations (Skew)

If the skew angle of the bent is 20° or less, the deck and soffit slab reinforcement are placed parallel to the centerline of bent cap. This bar reinforcement configuration allows the bent cap longitudinal reinforcement to be placed as far from the extreme compression fiber as possible to optimize flexural capacity. When the bent cap skew angle is greater than 20°, the deck and soffit slab reinforcement are typically placed perpendicular to the centerline of the bridge and the bent cap top longitudinal steel must be placed below the deck reinforcement or above the soffit reinforcement. Details of bent cap reinforcement for different skew angles can be found in BDD 7−45 and 7−43 (Caltrans, 1986).

12.5 DESIGN EXAMPLES

The section contains two design examples including a reinforced concrete integral bent cap and a drop bent cap to illustrate the main design process of using the AASHTO LRFD Bridge Design Specifications (AASHTO, 2012) and California Amendments (Caltrans, 2014). Figure 12.5-1 shows a general design flowchart for a bent cap.

It should be noted that the examples do not constitute a complete bent cap design. Only selected work has been done to demonstrate design methods. For example, tension steel has not been designed for every span of the bent cap as it would be done for an actual bent cap design. Additionally, there are other design considerations not considered in examples. For instance, seismic design will not been addressed. It is hoped, however, that the example will provide good foundation for design of bent caps.

The information contained in this section should not be used as a design guide in place of reading the specifications. There are often several ways to solve a design problem. It is recommended that prior to applying any formula or procedure contained within this section, the designer should read the appropriate Articles of the Caltrans currently adopted AASHTO LRFD Bridge Design Specifications and California Amendments to be certain that the described formula or procedure is appropriate for use.

12.5.1 Integral Bent Cap

The integral bent cap example shown on the following page is used in a three-span cast-in-place prestressed concrete box girder bridge.

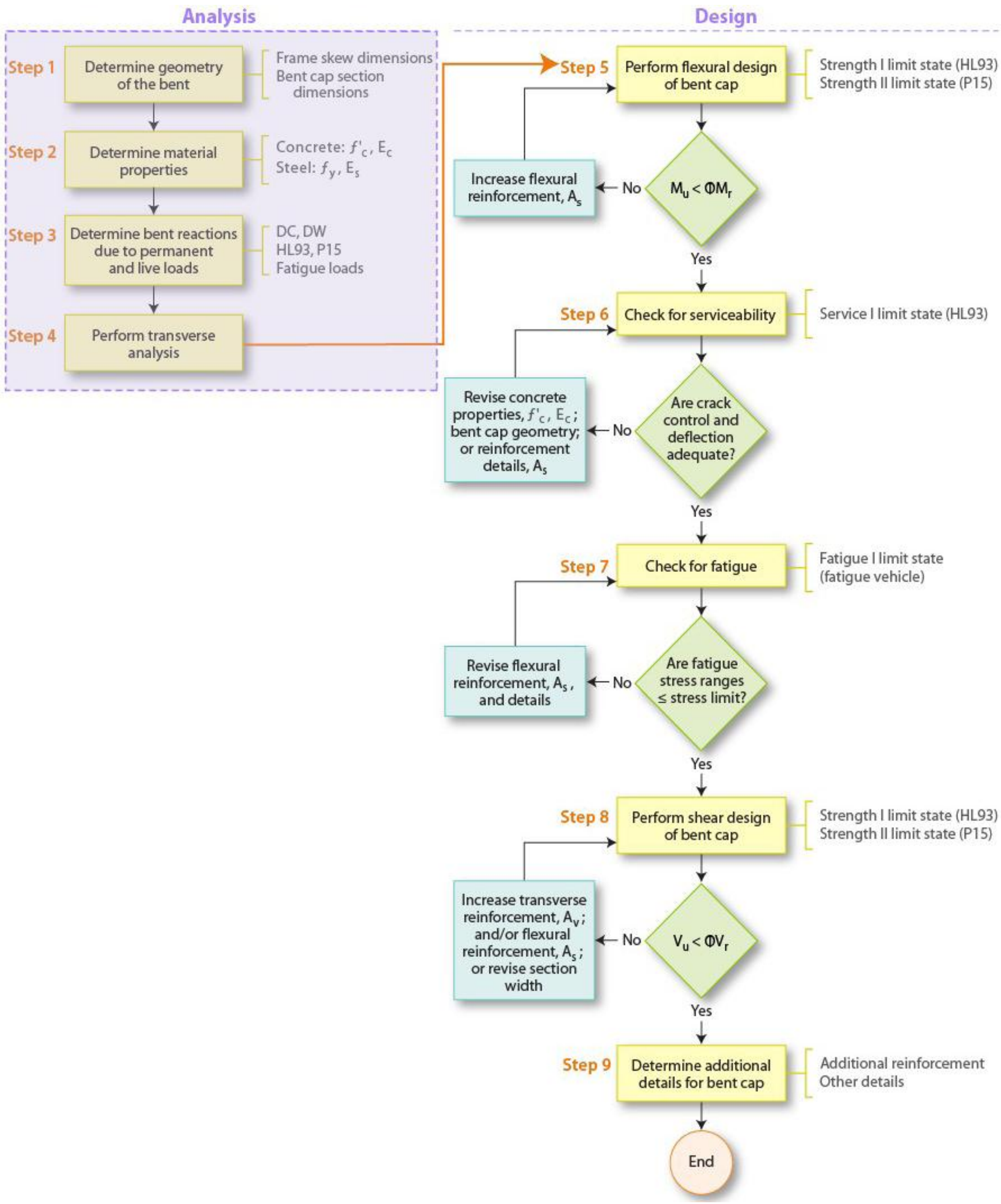


Figure 12.5-1 Bent Cap Analysis and Design Flowchart

12.5.1.1 Bent Cap Data

Integral bent cap is supported by two column bent as shown in Figure 12.5-2. The box girder has four cells of depth 6.75 ft with a skew angle is 20°. Concrete column diameter is 6 ft. diameter columns.

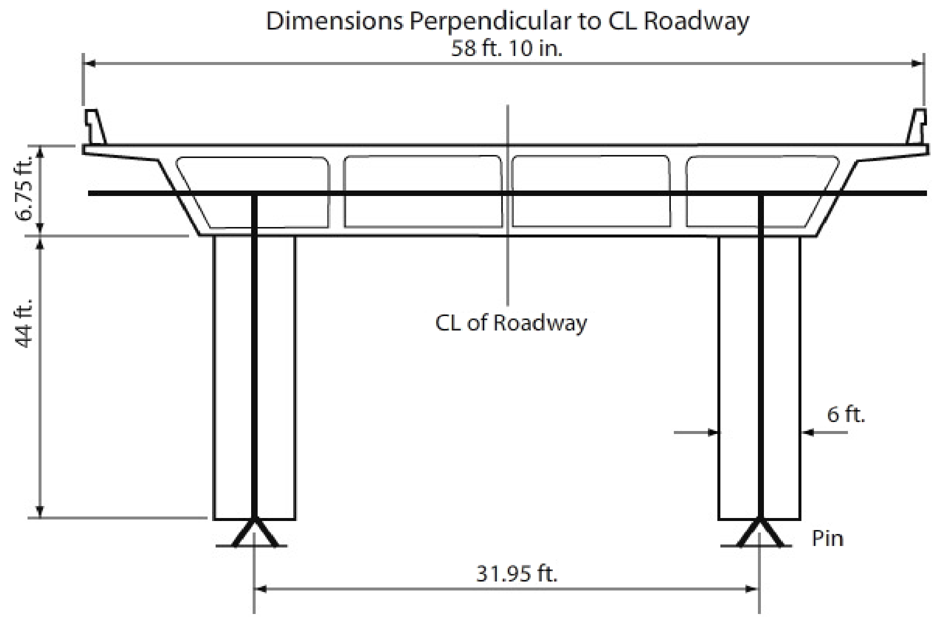


Figure 12.5-2 Two Column Bent

Note: These dimensions are perpendicular to the centerline (CL) of the roadway and are not actual dimensions of the bent since the bent is skewed at an angle to the roadway.

12.5.1.2 Design Requirements

Perform the structural analysis, flexural, and shear design as shown in Figure 12.5-1 in accordance with the AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO, 2012) with California Amendments (Caltrans, 2014).

12.5.1.3 Step 1: Determine Geometry of Bent

Bent dimensions along the skew are shown in Figure 12.5-3. Ranges of bent cross section types are shown in Figure 12.5-4.

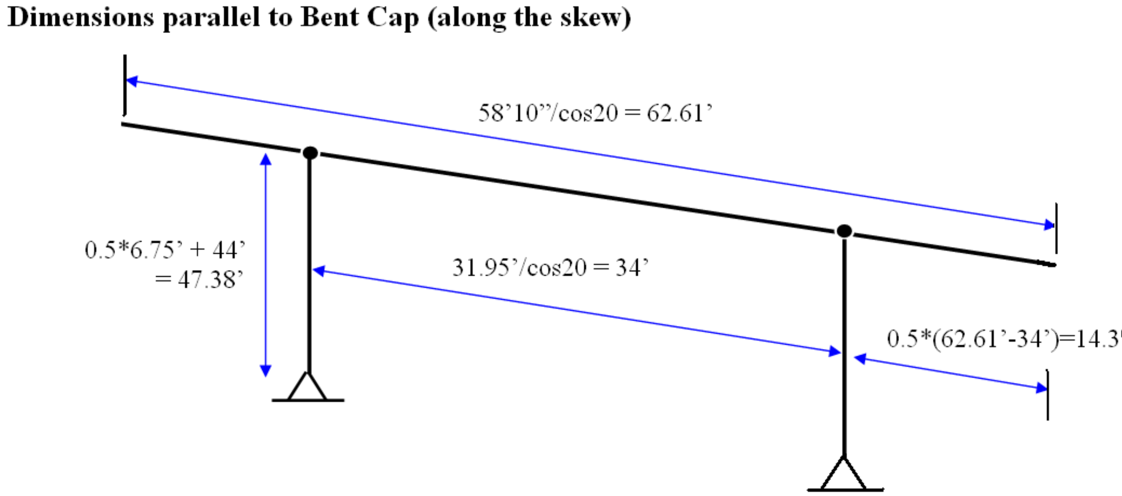


Figure 12.5-3 Bent Dimensions along the Skew

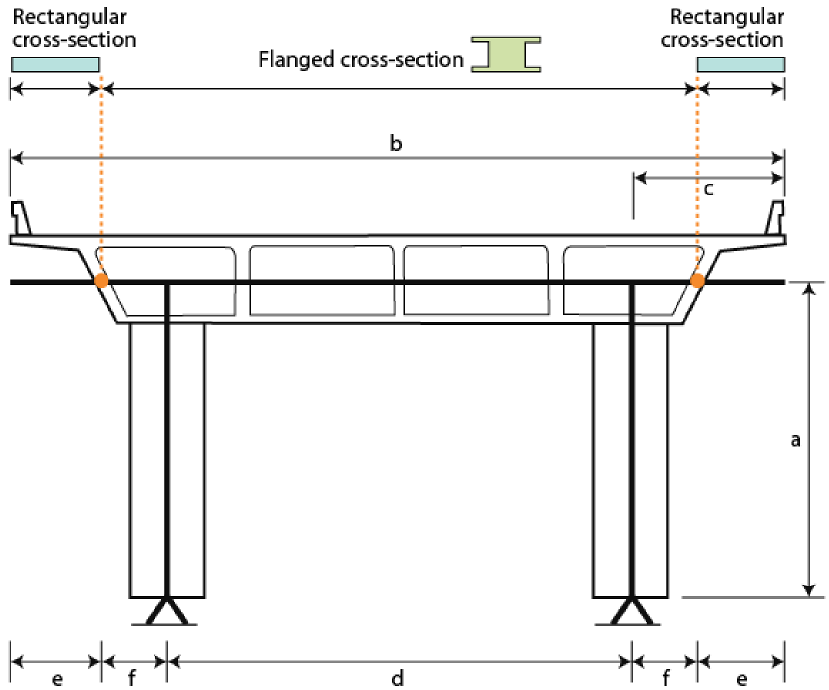


Figure 12.5-4 Bent Cross Section Types

where :

a = distance from the centroid of gravity of the superstructure to the bottom of column

b = top width of superstructure

c = distance from the CL of the columns to the edge of deck (*e* + *f*)

d = distance between the CL of columns

e = distance from the CL of exterior girder to the edge of deck (EOD)

f = distance from the CL of the column to the CL of the exterior girder

For the example bridge, assume that CG of the bent cap is at mid-depth of the cap beam.

12.5.1.3.1 Rectangular Cross Section

The rectangular cross section spans from the CL of the exterior girder to the EOD. These sections are assumed as rectangular in order to simplify the stick model.

The dimension e in the geometric model is the distance from the CL of exterior girder to the EOD along the skew, as shown in Figure 12.5-5.

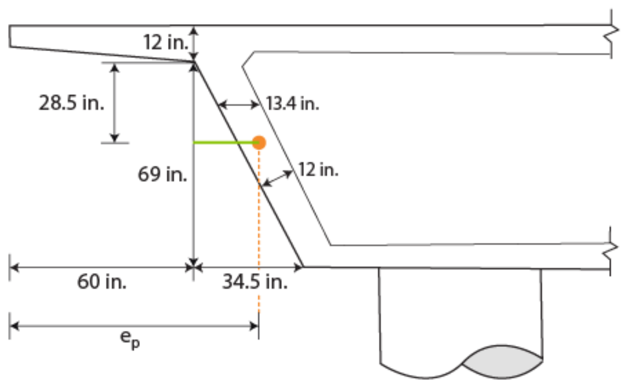


Figure 12.5-5 Rectangular Cross Section

$$e_p = \frac{1}{2}(13.4) + \left[\frac{28.5}{69}(34.5)\right] + 60 = 81 \text{ in.}$$

$$e \text{ (alongskew)} = \frac{81}{\cos(20)} = 86.2 \text{ in.} = 7.18 \text{ ft}$$

$$f = c - e = 14.3 - 7.18 = 7.12 \text{ ft}$$

12.5.1.3.2 Flanged Cross Section

Range of flanged cross section is shown in Figure 12.5-6.

Minimum bent cap width = column width + 2 ft (SDC 7.4.2.1)

Width of bent cap, B_{cap} = 6 + 2 = 8 ft

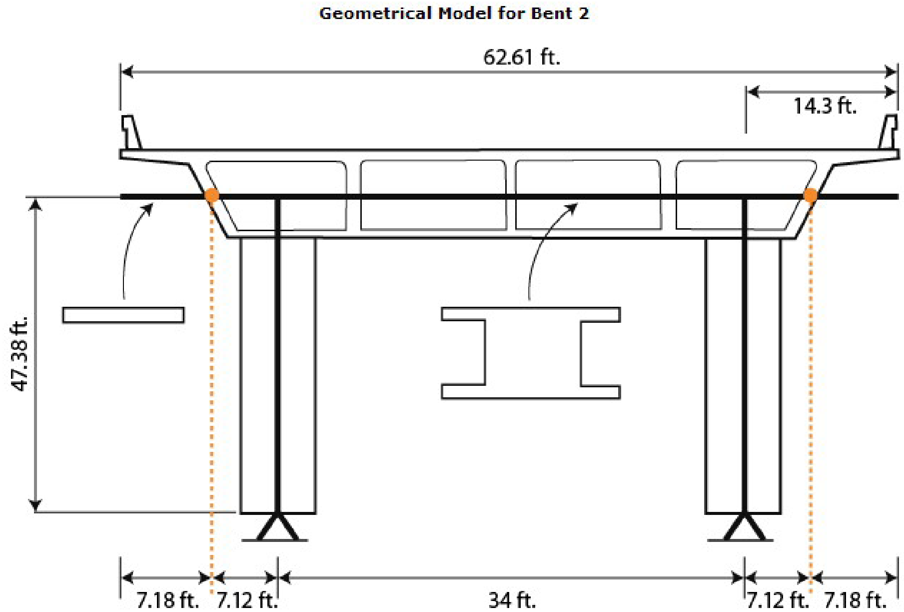


Figure 12.5-6 Range of Flanged Cross Section

Effective bent cap flange or overhang width = width of bent cap + 2 (width of overhang at each side), as shown in Figure 12.5-7.

Width of overhang is: (AASHTO 4.6.2.6.5)

Least of -6 ×(least soffit slab thickness) = 6 (8.25) = 49.5 in.

0.1×(span length of the bent cap) = 0.1(34) = 3.4 ft = 40.8 in

0.1×(2 × length of cantilever span) = 0.1 (2×14.3) = 2.86 ft = 34.3 in.

Total effective flange width = 8 + 2.86 + 2.86 = 13.72 ft = 164.64 in.

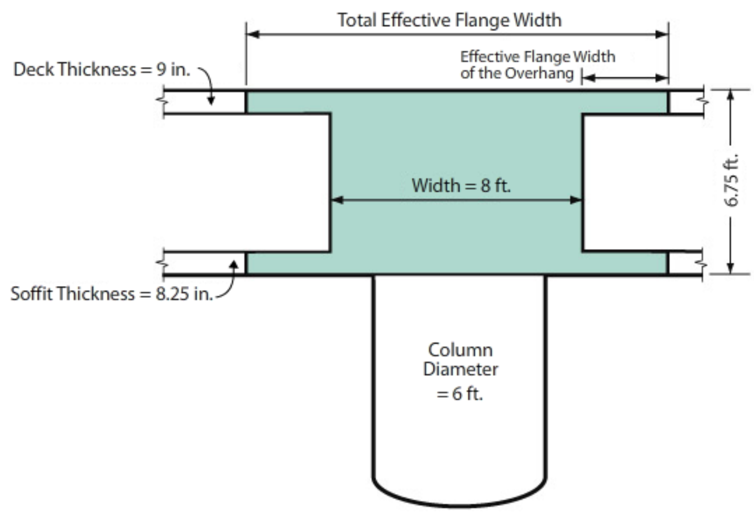


Figure 12.5-7 Effective Flange Width

12.5.1.4 Step 2: Determine Material Property

A706 Steel reinforcement with f_y = 60 ksi and E_s = 29,000 ksi, and concrete with f'_c = 4 ksi and E_c = 3,645 ksi, are used in design example.

12.5.1.5 Step 3: Determine Bent Reactions due to Permanent and Live Load

Bent reactions due to permanent and live load are calculated as follows.

12.5.1.5.1 Reactions due to Permanent Loads

The permanent loads are comprised of:

- Dead load of structural components and non-structural attachments (DC)
- Dead load of wearing surfaces and utilities (DW)
- Self-weight of bent cap

The unfactored bent reactions for DC and DW obtained from a longitudinal analysis are shown in Table 12.5-1.

12.5.1.5.2 Self-Weight of the Bent Cap

The self-weight of the bent cap is modeled as a uniformly distributed load as shown in Figure 12.5-8.

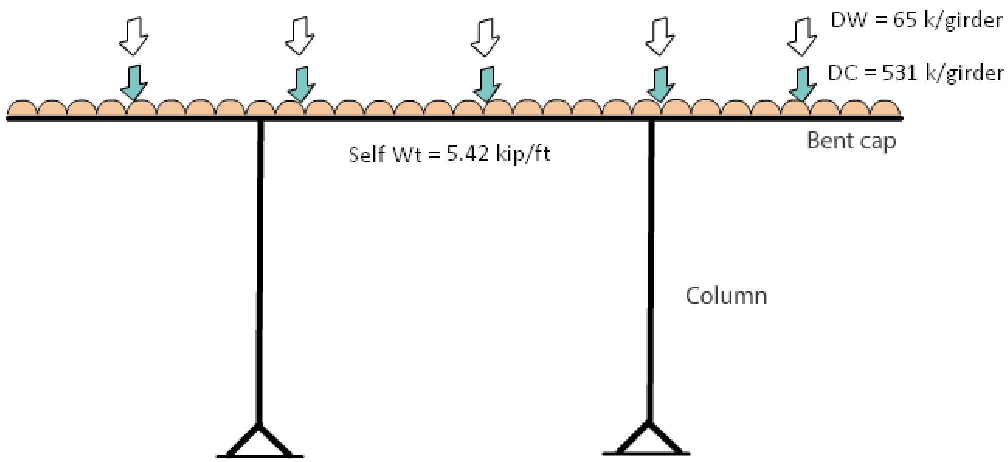


Figure 12.5-8 Permanent Load on Bent Cap

Self-weight of bent cap = (cross-sectional area of the bent cap solid section) × (average length of bent cap) × (unit weight of concrete)

Average length = 7.12 + 34 + 7.12 = 48.24 ft

Box cell area = 217.9 ft^2

Bent cap self-weigh = (217.90) (8) (0.15) = 261.48 kips

Self-weight modeled as uniform load = 261.48 / 48.24 = 5.42 kips/ft

Note: While calculating the self-weight of the bent cap, be careful not to include the portion of the deck, soffit slab, and girder thicknesses at the bent cap if they have already been included in the longitudinal analysis of the bridge.

Table 12.5-1 Unfactored Reactions due to DC, DW, HL-93, P-15, and Fatigue Vehicles

	Bent Reactions (kip)	Dynamic Allowance Factor	Final Bent Reactions (kip)
DC	2,651.5	NA	2,651.5
DW	325	NA	325
Self-Weight	261.48	NA	261.48
HL-93 Vehicle Truck Lane	114.82 99.34	1.33 1	252
Permit Vehicle	360.77	1.25	451
Fatigue Vehicle	70.66	1.15	81.25

For both DC and DW, these bent reactions obtained from the longitudinal analysis are modeled as concentrated load acting at the CL of each girder framing into the bent cap.

- Reaction due to DC on the bent cap for each girder (kip)

  = 2651.5 / 5 = 531 kips

- Reaction due to DW on the bent cap for each girder (kip)

  = 325 / 5 = 65 kips

12.5.1.5.3 Live Load as Two Wheels

Vehicular live loads as two wheels are applied on the bent caps as shown in Figure 12.5-9.

- HL-93 vehicle: 252 / 2 = 126 kips
- Permit vehicle: 451 / 2 = 226 kips
- Fatigue vehicle : 82 / 2 = 41 kips

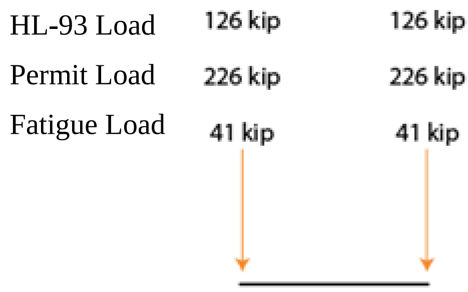


Figure 12.5-9 Bent Reaction due to Live Loads

The HL-93 vehicle live loads can be one, two, three, or four lanes.

Figure 12.5-10 shows different HL-993 truck placement for transverse analysis using CSiBridge. Figure12.5-11 shows placement of two HL-93 lanes.

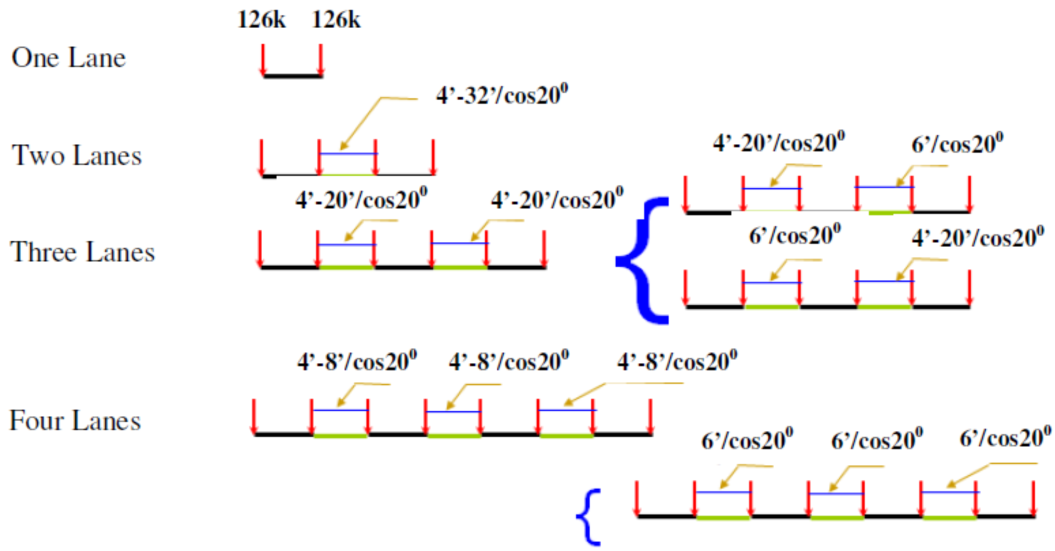


Figure 12.5-10 Possible Scenarios for Placement of Live Load

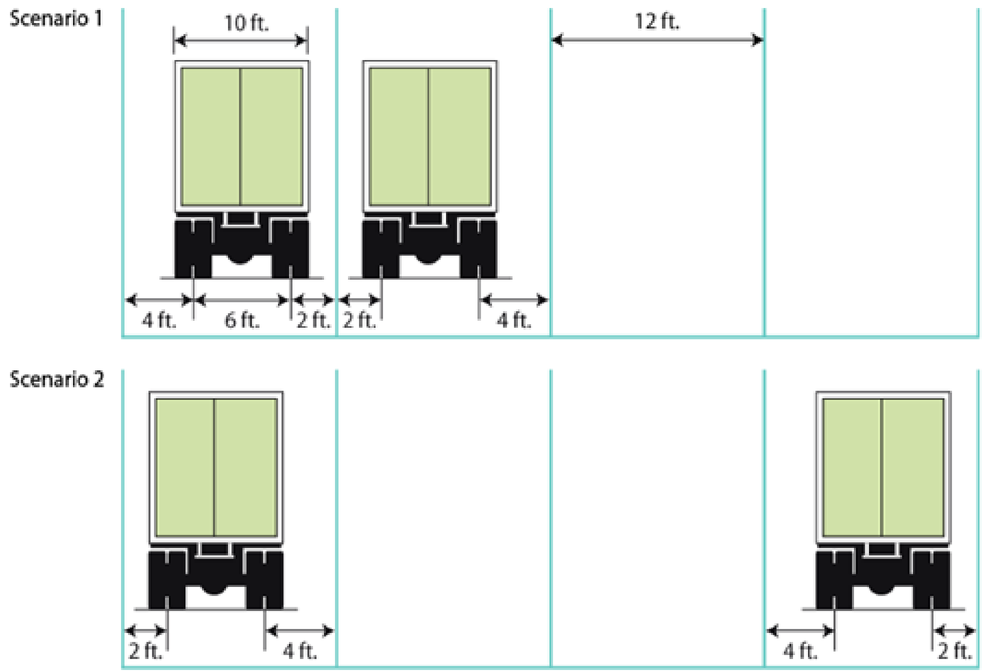


Figure 12.5-11 Two Scenarios for Two Truck Case

Permit vehicle live load may be only one or two lanes as shown in Figure 12.5-12.

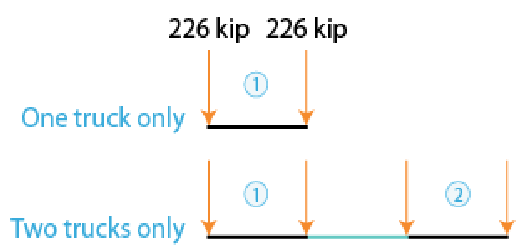


Figure 12.5-12 Permit Trucks

Fatigue vehicle live load is only one truck as shown in Figure 12.5-13.

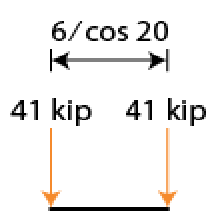


Figure 12.5-13 Fatigue Truck

12.5.1.6 Step 4: Perform Transverse Analysis

The goal of the transverse analysis is to obtain force effect envelopes of all possible live load cases.

The example bridge has four design lanes. Each of the different live load trucks can be placed in these design lanes. The number and placement of these trucks depends on the type of load:

- HL-93 vehicle
- Permit truck
- Fatigue truck

Since there may be many live load cases to consider, computer software is usually used to determine:

- Maximum moment (negative or positive) and associated shear
- Maximum shear and associated moment

Available computer programs for transverse analysis are:

- CSiBridge
- VBENT
- LEAP Bridge - RCPIER
- CTBridge

For this example bridge, results are obtained by performing transverse analysis with CSiBrdge.

The results from the transverse analysis are obtained at every tenth point or at selected points. These results are shown separately for DC, DW, HL-93, permit, and fatigue loads. Table 12.5-2 and 12.5-3 list the controlling unfactored moments.

Note: Bent transverse analysis using CsiBridge is not covered under this chapter. Please refer to the following URL for step-by-step procedures to generate the bent cap model in CSiBridge for transverse analysis:

http://onramp.dot.ca.gov/hq/des/sd/SD_training/intro.html

12.5.1.7 Step 5: Perform Flexural Design

The design equation is as follows:

$$M_u \leq M_r = \phi M_n \tag{12.5-1}$$

12.5.1.7.1 Calculate Controlling Unfactored Moment

Table 12.5-2 shows unfactored controlling moments including impact for the bent cap, obtained from the transverse analysis of the bridge using CSiBridge.

Table 12.5-2 Unfactored Moments

Load and Moment	Moment at Mid-Span (kip-ft)	Moment at Face of Column (kip-ft)
DC, M_{DC}	3,377	-1,760
DW, M_{DW}	339	-217
HL-93 Vehicle, $M_{HL\text{-}93}$	2,683	-1,859
Permit Vehicle, $M_{P\text{-}15}$	4,571	-3,336

12.5.1.7.2 Check Positive Moment at Midspan

Factored moments are as follows:

- Strength I

$$M_u = 1.25M_{DC} + 1.5M_{DW} + 1.75M_{HL-93}$$
$$= (1.25)(3{,}377) + (1.5)(339) + (1.75)(2{,}683) = 9{,}425 \text{ kip-ft}$$

- Strength II

$$M_u = 1.25M_{DC} + 1.5M_{DW} + 1.35M_{P15}$$
$$= (1.25)(3{,}377) + (1.5)(339) + (1.35)(4{,}571) = 10{,}901 \text{ kip-ft}$$

For the example bent cap, f_{cpe} = 0. The bent cap is designed for the monolithic section to resist all loads. substitute S_{nc} for S_c and cracking moment is calculated by:

$$M_{cr} = \gamma_3 \left((\gamma_1 f_r) S_c \right) \quad (12.5\text{-}2)$$

$$f_r = 0.24\sqrt{f'_c} = (0.24)\sqrt{4} = 0.48 \text{ ksi} \quad \text{(AASHTO 5.4.2.6)}$$

γ_1 = 1.60 (for concrete structure, AASHTO 5.7.3.3.2)

γ_3 = 0.75 (for A706, Grade 60 reinforcement, AASHTO 5.7.3.3.2)

I_g (flanged section) = 280.5 ft^4; Y_t = 40.30 in.; Y_b = 40.70 in.

$$S_c = \frac{I_g}{Y_b} = \frac{280.5}{(40.70/12)} = 82.70 \text{ kip-ft}$$

$$M_{cr} = S_c f_r = 0.75[(1.60)(0.48)(82.70)](12)^2 = 6{,}860 \text{ kip-ft}$$

$$M_{u(\min)} = \text{Lesser of } \begin{Bmatrix} 1.0M_{cr} \\ 1.33M_u \end{Bmatrix}$$

$$= \text{Lesser of } \begin{Bmatrix} 1.0(6{,}860) = 6{,}860 \text{ kip-ft} \\ 1.33(10{,}901) = 14{,}498 \text{ kip-ft} \end{Bmatrix} = 6{,}860 \text{ kip-ft}$$

Therefore, the controlling factored moment at mid-span is:

M_u = 10,901 kip-ft

For the bent section without prestressing steel and with neglecting the effect of compression reinforcement, the M_n equation reduces to:

$$M_n = A_s f_s\left(d_s - \frac{a}{2}\right) + 0.85 f'_c\left(b - b_w\right)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \qquad (12.5\text{-}2)$$

Assuming the neutral axis lies in the compression flange (rectangular section behavior), nominal flexural resistance M_n is calculated by:

$$M_n = A_s f_s\left(d_s - \frac{a}{2}\right) \qquad (12.5\text{-}3)$$

where:

M_n = nominal flexural resistance (kip-in.)

f_s = 60 ksi

d_s = 81 – (5.70 + 1.63) = 73.67 in. (assuming vertical bundles of #11)

b = 164.64 in.

b_w = 96 in.

h_f = 9 in.

ϕ = 0.9 (AASHTO 5.5.4.2)

a = $c\,\beta$

where:

$\beta = 0.85$ (AASHTO 5.7.2.2)

$$a = \left(\frac{A_s f_s}{0.85 f'_c b}\right)$$

$$c = \left(\frac{A_s f_s}{0.85 f'_c \beta b}\right)$$

Rearrange and substitute design parameters into Eq. (12.5-1) to obtain:

$$\frac{M_u}{0.9} \le A_s f_s \left[d_s - \left(\frac{A_s f_s}{0.85 f'_c b} \right) \left(\frac{1}{2} \right) \right]$$

$$\frac{10{,}910(12)}{0.9} \le A_s (60) \left[73.67 - \left(\frac{A_s (60)}{0.85(4).(164.64)} \right) \left(\frac{1}{2} \right) \right]$$

Solving for A_s for positive moment region

$A_s \quad \ge$ 33.71 in.^{2}

Provide 22- #11 as bottom reinforcement: A_s = 34.32 in.^{2}

$$c = \left(\frac{A_s f_s}{0.85 f'_c \text{b} b} \right) = \frac{34.32(60)}{(0.85)(4)(0.85)(164.64)} = 4.33 \text{in.} < 9 \text{ in.}$$

Assumption of the neutral axis within in the flange is correct.

$$M_r = 0.9 M_n = 0.9 (A_s f_y) \left(d_s - \frac{c\beta}{2} \right)$$

$$= 0.9(34.32)(60) \left(73.67 - \frac{(4.33)(0.85)}{2} \right)$$

$$= 133{,}120 \text{ kip-in.}$$

$$= 11{,}093 \text{ kip-ft} \qquad > M_u = 10{,}901 \text{ kip-ft}$$

O.K.

Strain diagram is shown in Figure 12.5-14:

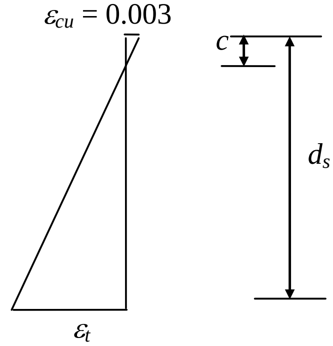


Figure 12.5-14 Strain Diagram

$$\left(\frac{\varepsilon_{cu}}{c} \right) = \left(\frac{\varepsilon_t}{d_s - c} \right)$$

$$\left(\frac{0.003}{4.33}\right)=\left(\frac{\varepsilon_t}{73.67-4.33}\right)$$

Since $\varepsilon_t = 0.048 > 0.005$, assumption of $\phi = 0.90$ is correct.

12.5.1.7.3 Check Negative Moment at the Face of Column

Factored moments are as follows:

- Strength I

$$M_u = 1.25M_{DC} + 1.5M_{DW} + 1.75M_{HL-93}$$
$$= (1.25)(-1{,}760) + (1.5)(-217) + (1.75)(-1{,}859) = -5{,}779 \text{ kip-ft}$$

- Strength II

$$M_u = 1.25M_{DC} + 1.5M_{DW} + 1.35M_{HL-93}$$
$$= (1.25)(-1{,}760) + (1.5)(-217) + (1.35)(-3{,}336) = -7{,}029 \text{ kip-ft}$$

For the example bent cap, f_{cpe} = 0. Also, the bent cap is designed for the monolithic section to resist all loads, substitute S_{nc} for S_c and cracking moment is calculated by:

$$M_{cr} = \gamma_3 \left((\gamma_1 f_r) S_c \right) \qquad (12.5\text{-}2)$$

where :

$f_r = 0.24\sqrt{f_c'} = (0.24)\sqrt{4} = 0.48$ ksi (AASHTO 5.4.2.6)

$\gamma_1 = 1.60$ (for concrete structure, AASHTO 5.7.3.3.2)

$\gamma_3 = 0.75$ (for A706, Grade 60 reinforcement, AASHTO 5.7.3.3.2)

I_g (flanged section) = 280.5 ft^4; Y_t = 40.30 in.; Y_b = 40.70 in.

$$S_c = \frac{I_g}{Y_b} = \frac{280.5}{(40.30/12)} = 83.52 \text{ ft}^2$$

$$M_{cr} = S_c f_r = 0.75[(1.60)(0.48)(83.52)](12)^2 = 6{,}928 \text{ kip-ft}$$

$$M_{u(\min)} = \text{Lesser of } \begin{Bmatrix} 1.0M_{cr} \\ 1.33M_u \end{Bmatrix}$$

$$= \text{Lesser of } \begin{Bmatrix} 1.0(6{,}928) = 6{,}928 \text{ kip-ft} \\ 1.33(7{,}029) = 9{,}349 \text{ kip-ft} \end{Bmatrix} = 6{,}928 \text{ kip-ft}$$

Therefore, the controlling factored negative moment at the face of column is:

$$|M_u| = 7{,}029 \text{ kip-ft}$$

Assuming the neutral axis is within the compression flange (rectangular section behavior), nominal flexural resistance M_n is calculated by:

$$M_n = A_s f_s \left(d_s - \frac{a}{2} \right)$$

where:

f_s = 60 ksi

b_w = 96 in.

b = 164.64 in.

d_s = 81 – (5 + 1.63) = 74.37 in. (assuming vertical bundles of #11)

ϕ = 0.90 (AASHTO 5.5.4.2)

$a = c\,\beta$ where $\beta = 0.85$ (AASHTO 5.7.2.2)

Rearrange and substitute design parameters into Eq. (12.5-1) to obtain:

$$\frac{M_u}{0.9} \le A_s f_y \left(d_s - \left(\frac{A_s f_y}{0.85 f_c' b} \right)\left(\frac{1}{2} \right) \right)$$

$$\frac{7{,}029(12)}{0.9} \le A_s(60)\left(74.37 - \left(\frac{A_s(60)}{0.85(4)(164.64)} \right)\left(\frac{1}{2} \right) \right)$$

Solving for A_s for negative moment at the face of the support:

A_s = 21.33 in.^{2}

Provide 14 - #11 as Top reinforcement (A_s = 21.84 in.^{2})

$$c = \frac{A_s f_s}{0.85 f_c' \beta b} = \frac{(21.84)(60)}{(0.85)(4)(0.85)(161.64)} = 2.75 \text{ in.} < 8.25\text{in.}$$

Assumption of the neutral axis within the flange, is correct

$$M_r = \phi M_n = 0.9(A_s f_y)\left(d_s - \frac{c\beta}{2} \right)$$

$$= 0.9(21.84)(60)\left(74.37 - \frac{2.75(0.85)}{2} \right)$$

$$= 86{,}331 \text{ kip-in.}$$

$$= 7{,}194 \text{ kip-ft} \quad > |M_u| = 7{,}029 \text{ kip-ft}$$

O.K.

Strain in steel:

$$\left(\frac{\varepsilon_{cu}}{c}\right)=\left(\frac{\varepsilon_t}{d_s-c}\right)$$

$$\left(\frac{0.003}{2.75}\right)=\left(\frac{\varepsilon_t}{74.37-2.75}\right)$$

Therefore assumption of $\phi = 0.90$ is correct.

12.5.1.8 Step 6: Check for Serviceability

Cracks occur whenever the tension in the gross section exceeds the cracking strength (modulus of rupture) of concrete. One can control or avoid flexural cracking in a concrete component by providing tension reinforcement at certain specified spacing.

The spacing, *s*, of mild steel reinforcement in the layer closest to the tension face:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(AASHTO 5.7.3.4-1)}$$

in which:

$$\beta_s = 1+\frac{d_c}{0.7(h-d_c)}$$

where:

γ_e = exposure factor taken as 0.75 by considering Class 2 exposure condition (CA 5.7.3.4)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)
= 1.5 + 0.69 + 0.69 / 2 = 2.54 in.

h = overall thickness or depth of the component (in.) = 81 in.

f_{ss} = tensile stress in steel reinforcement at the service time limit state (ksi)

Tensile stress in steel reinforcement may be calculated based on the transformed section by the following procedure valid for both rectangular and flanged sections (Figure 12.5-15).

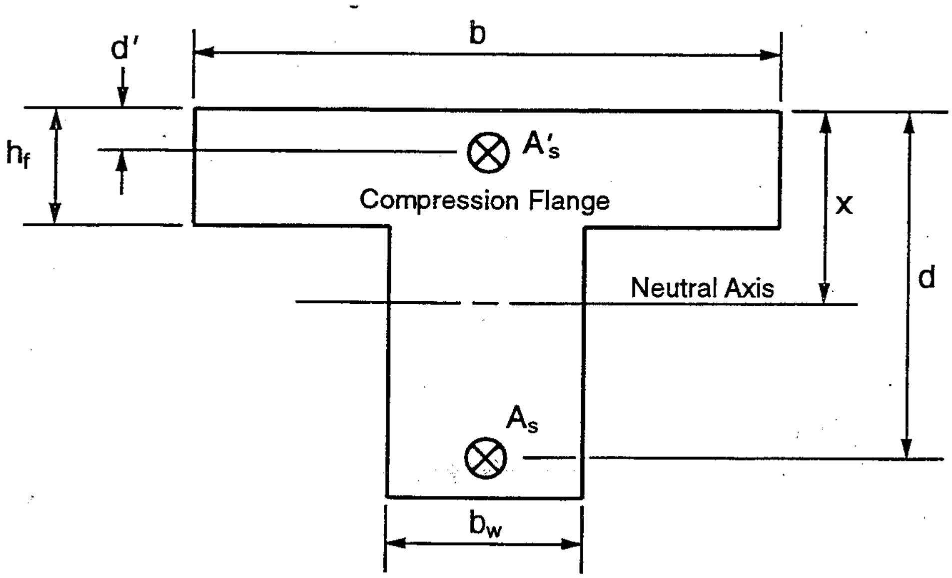


Figure 12.5-15 A Typical Flanged Section

Note: Given b, b_w, h_f, d, d', A_s, A'_s, $n = E_s / E_c$, M = applied moment

If $h \neq 0$ and $b \geq \dfrac{2}{h_f^2}\left[n\left(d - h_f\right)A_s - (n-1)\left(h_f - d'\right)A'_s\right]$, then set $b_w = b$

Set $\quad B = \dfrac{1}{b_w}\left(h_f\left(b - b_w\right) + nA_s + (n-1)A'_s\right)$

Set $\quad C = \dfrac{2}{b_w}\left(h_f^2\left(b - b_w\right)/2 + ndA_s + (n-1)d'A'_s\right)$

$x = \sqrt{B^2 + C} - B$ (assumes $x \geq d'$)

$$I = \frac{1}{3}bx^3 - \frac{1}{3}\left(b - b_w\right)\left(x - h_f\right)^3 + nA_s\left(d - x\right)^2 + \left(n - 1\right)A'_s\left(x - d'\right)^2$$

$f'_c = \dfrac{Mx}{I}$ = stress in top fiber of compression flange

$f'_s = \dfrac{nM\left(x - d'\right)}{I} = nf'_c\left(1 - \dfrac{d'}{x}\right)$ = stress in compression steel

$f_s = \dfrac{nM\left(d - x\right)}{I} = nf'_c\left(\dfrac{d}{x} - 1\right)$ = stress in tension steel

12.5.1.8.1 Bottom Reinforcement

For service load combination, the permit loads are not considered.

M_{ser} = 3,377 + 339 + 2,683 = 6,399 kip-ft

Bent cap section:

b_e = 13.72 ft = 164.64 in.

b_w = 96 in.

h_f = 9.125 in.

E_s = 29,000 ksi

E_c = 3,645 ksi

n = E_s / E_c = 7.96

Assume #16-5 as crack control reinforcement.

Effective, A_{s1} = (0.31)(16) cos (20°) = 4.66 in.^{2}

Tension reinforcement (bottom), A_{s2} = 1.56 (22) = 34.32 in.^{2}

Compression reinforcement (top), A'_s = 1.56 (14) = 21.84 in.^{2}

where:

d_{e1} = effective depth from extreme comp fiber to the centroid of the crack control reinforcement

= 81 – (1.5 + 0.69 + 0.69/2) = 78.47 in.

d_{e2} = effective depth from extreme comp fiber to the centroid of tension reinforcement (bottom)

= 81 – (5.7 + 1.63) = 73.67 in.

d' = effective depth from extreme comp fiber to the centroid of compression reinforcement (top)

= 5 + 1.63 = 6.63 in.

If $\sum\left(\dfrac{b_e h_f^2}{2} + (n-1)A'_s\left(h_f - d'\right)\right) > \sum\left(nA_s\right)\left(d - h_f\right)$, then neutral axis, y, lies in the flange.

$$\sum\left(\frac{(164.64)(9)^2}{2} + (7.96 - 1)(21.84)(9 - 6.63)\right) = 7{,}028$$

< (7.96) (4.66) (78.47 – 9) + (7.96) (34.32) (73.67 – 9) = 20,244

Therefore, the neutral axis is within web and the compression block has T-section shape.

$$\sum\left(\left(b_e - b_w\right)h_f\left(y - \frac{h_f}{2}\right) + \frac{b_w y^2}{2} + \left(n-1\right)A_s'\left(y - d'\right)\right) = \sum\left(nA_s\right)\left(d - y\right)$$

(164.64 - 96) (9) (y – 9/2) + 96 (y^2/2) + (7.96 – 1) (21.84) (y – 6.63)

= 7.96 (4.66) (78.47 – y) + 7.96 (34.32) (73.67 - y)

Solving for y:

y = 14.93 in.

$$I = \frac{b_e y^3}{3} - \frac{\left(b_e - b_w\right)\left(y - h_f\right)^3}{3} + \left(n-1\right)A_s'\left(y - d'\right)^2 + \sum\left(nA_s\right)\left(d - y\right)^2$$

$$= \frac{164.64(14.93)^3}{3} - \frac{(164.64 - 96)(14.93 - 9.0)^3}{3} + (7.96 - 1)(21.84)(14.93 - 6.63)^2$$

$$+\left[7.96(4.66)(78.47 - 14.93)^2 + 7.96(34.32)(73.67 - 14.93)^2\right]$$

$$= 1{,}289{,}972 \text{ in.}^4 = 62.21 \text{ ft}^4$$

$$f_{ss} = \frac{nM(d-y)}{I} = \frac{7.96(6{,}399)(12)(78.47 - 14.93)}{62.21(12)^4} = 30.6 \text{ ksi}$$

$$\beta_s = 1 + \frac{2.54}{0.7\left(81 - 2.54\right)} = 1.046$$

$$s \le \frac{700(0.75)}{1.046(30.6)} - 2\left(2.54\right) = 11.32 \text{ in.}$$

12.5.1.8.2 Top Reinforcement

Calculations for crack reinforcement at top of bent cap are not shown. Designer can use WinCONC or similar computer program to check serviceability of section.

Flexural reinforcements are shown in Figure 12.5-16.

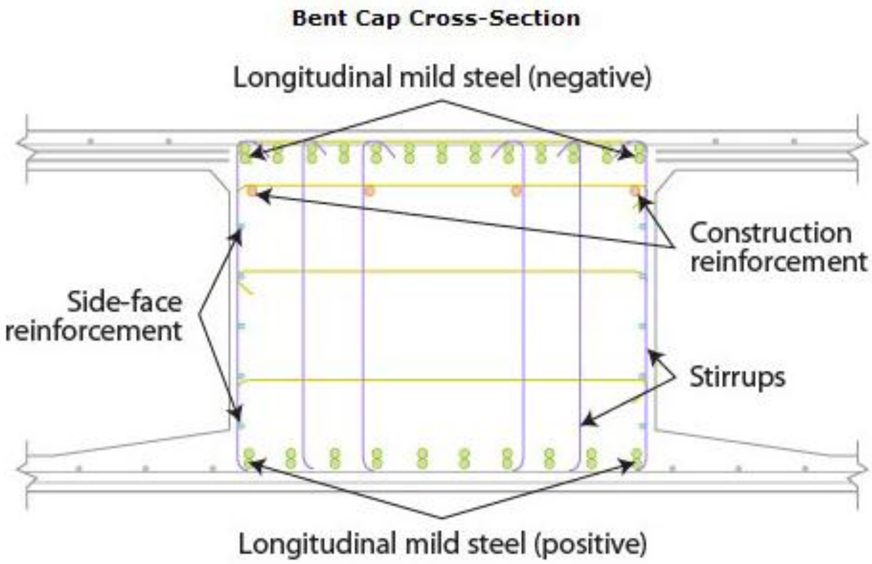


Figure 12.5-16 Bent Cap Cross Section

12.5.1.9 Step 7: Check for Fatigue

Unfactored fatigue moments at midspan are listed in Table 12.5-3.

Table 12.5-3 Unfactored Fatigue Load Moments at Midspan.

Load	Max. and Min. Moment at Mid-span (kip-ft)		Max. and Min. Moment at Face of Column (kip-ft)	
	+Positive	-Negtive	+Positive	-Negative
DC	3,377	0	0	-1,760
DW	339	0	0	-217
Fatigue Vehicle I	789	-264	144	-504

For Fatigue I load combination, a load factor of 1.75 shall be used.

$M_{u(max)}$ = 3,377 + 339 + 1.75(789) = 5,096.8 kip-ft

$M_{u(min)}$ = 3,377 + 339 + 1.75(-264) = 3,254 kip-ft

I = 69.5 ft^4, y = 14.96 in.

$$f_s = \frac{nM(d-y)}{I}$$

$f_{s(max)}$ = 7.96 (5,096.8)(12) (78.47 – 14.96)/(69.5)(12^4) = 21.45 ksi

$f_{s(min)}$ = 7.96 (3,254)(12) (78.47 – 14.96)/(69.5)(12^4) = 13.70 ksi

$\gamma(\Delta f) = f_{s(max)} - f_{s(min)} = 21.45 - 13.70 = 7.75$ ksi

$(\Delta F)_{TH} = 24 - 0.33 f_{min} = 24 - 0.33(13.7) = 19.48$ ksi

$\gamma(\Delta f) = 7.75 \text{ ksi} < (\Delta F)_{TH} = 19.48$ ksi OK

Fatigue requirement at the midspan is met.

The following critical locations shall be checked for fatigue requirement:

- At maximum difference in positive moment region (bottom steel)
- At maximum difference in negative moment region (top steel)
- Moment reversal (top and bottom steel)

Calculations for fatigue check at top of bent cap not shown. Designer can use WinCONC or similar computer program to check fatigue limit state of the section.

12.5.1.10 Step 8: Perform Shear Design

The shear design equation is as follows:

$$V_u \leq V_r = \phi V_n \quad (12.5\text{-}2)$$

Shear design is performed for critical section located at the face of support.

12.5.1.10.1 Calculate Factored Shear

Table 12.5-4 lists unfactored shears including impact from CSiBridge analysis:

Table 12.5-4 Unfactored Shear at Face of Support

Load	Max. Shear (kip)	Assoc. Moment (kip-ft)	Max. Moment (kip-ft)	Assoc Shear (kip)
DC	-888	-1760	-1760	-888
DW	-98	-217	-217	-98
Design Vehicle	-338	588	-1860	-66
Permit Vehicle	-607	1054	-3335	-119

- Strength I: $V_{u(max)}$

  V_u = 1.25(-888) + 1.5(-98) + 1.75(-338) = -1,849 kips

  $M_{u(assoc)}$ = 1.25(-1,760) +1.5 (-217) +1.75(558) = -1,549 kip-ft

- Strength II: $V_{u(max)}$

  V_u = 1.25(-888) + 1.5(-98) + 1.35(-607) = -2,076 kips

  $M_{u(assoc)}$ = 1.25(-1,760) +1.5(-217) +1.35(1,054) = -1,102 kip-ft

12.5.1.10.2 *Determine θ and β*

Using AASHTO-LRFD Table B5.2-1 for sections with minimum amount of transverse reinforcement.

b_v = 8 ft = 96 in.

d_v, using results from flexural analysis:

$$d_v = d_e - \frac{a}{2} = 74.37 - \frac{(0.85)(2.75)}{2} = 73.20 \text{ in.}$$

$$\because \ d_v = 73.20 \text{ in.} > \text{larger} \begin{cases} 0.9d_e = (0.9)(74.37) = 66.93 \text{ in.} \\ 0.72h = (0.72)(81) = 58.3 \text{ in.} \end{cases}$$

Use d_v = 73.2 in.

Shear stress:

$$v_u = \frac{V_u}{\phi b_v d_v} = \frac{2{,}076}{0.9(96)(73.2)} = 0.328 \text{ ksi}$$

Shear stress factor:

$$\frac{v_u}{f'_c} = \frac{0.328}{4} = 0.082$$

Determine ε_x at mid-depth

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2(E_sA_s + E_pA_{ps})} \qquad \text{(AASHTO 5.8.3.4.2-4)}$$

As there is no prestressing force and axial force in bent cap, the above equation reduces to:

$$\varepsilon_x = \frac{\left(\frac{M_u}{d_v} + 0.5V_u \cot\theta\right)}{2(E_sA_s)} \qquad (12.5\text{-}3)$$

A_s = area of fully developed steel on flexural tension side of member = 28.08 in.^{2}

Assuming 0.5cot θ = 1 and use absolute values for M_u and V_u for strain calculation.

$$\varepsilon_x = \frac{\left(\frac{(1{,}102)(12)}{73.2} + 2{,}076\right)}{2(29{,}000)(28.08)} = 0.001386 = 1.386(10)^{-3}$$

AASHTO Table B5.2-1 lists values of θ and β as function of shear stress factor v_u / f'_c and strain at mid-depth of the bent cap ε_x:

AASHTO Table B5.2-1 Values of θ and β for Sections with Transverse Reinforcement

v_u / f'_c	$\varepsilon_x \times 1{,}000$								
	≤–0.2	≤–0.1	≤–0.05	≤0	≤0.125	≤0.25	≤0.5	≤0.75	≤1
≤0.075	22.3 6.32	20.4 4.75	21 4.1	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.1	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.5	34 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37 2.13
≤0.15	21.6 2.88	23.3 2.79	24.2 2.78	25 2.72	26.9 2.6	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.6	28 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.2	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.4	30 2.34	30.8 2.14	32.3 1.86	34 1.73	35.7 1.64
≤0.25	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.7	34.3 1.58	35.8 1.5

From above table:

β = 2.18

θ = 36.7

12.5.1.10.3 Determine Shear Reinforcement

Concrete contribution to shear resistance:

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v = 0.0316(2.18)\sqrt{4}(96)(73.2) = 968 \text{ kips}$$

Demand on shear stirrups:

$$V_s = \frac{V_u}{\phi} - V_c = \frac{2{,}076}{0.7} - 968 = 1{,}339 \text{ kips}$$

Required shear stirrups:

$$V_s = 1{,}339 = \frac{A_v f_y d_v}{s}\cot\theta = \frac{A_v(60)(73.2)}{s}\cot(36.7^o)$$

$$\frac{A_v}{s} = 0.227 \text{ in.}^2\text{/in.}$$

Minimum shear reinforcement:

$$\left(\frac{A_v}{s}\right)_{\min} = 0.0316\frac{\sqrt{f'_c}}{f_y}b_v = 0.0316\frac{\sqrt{4}}{60}(96) = 0.1\text{ in.}^2\text{/in.}$$

Required stirrups spacing:

A_v = use six legs #5

A_v = 0.31 (6) = 1.86 in.^{2}

$$\text{Required } s = \frac{1.86}{0.227} = 8.19 \text{ in.}$$

Use s = 6 in.

12.5.1.10.4 *Check Maximum Spacing*

For $\frac{v_u}{f'_c} < 0.125$, $S_{max} = 0.8d_v \leq 18$ in.

For $\frac{v_u}{f'_c} \geq 0.125$, $S_{max} = 0.4d_v \leq 12$ in.

At this particular section:

$\frac{v_u}{f'_c} = \frac{0.328}{4} = 0.082$, S_{max} = 0.8(73.2) = 58.56 in. but not greater than 18 in.

Maximum spacing allowed = 18 in.

Six legs #6 at 6 in. at the face of column meet this requirement.

Note: Place bent cap stirrups parallel to girders. (BDD 7-45.1)

12.5.1.10.5 *Check Tenth Points along Bent Cap*

For V_u, M_u, θ, β , V_c, V_s, S, A_{vmin}, and S_{max}, Figure 12.5-17 shows stirrup spacing along the bent cap length.

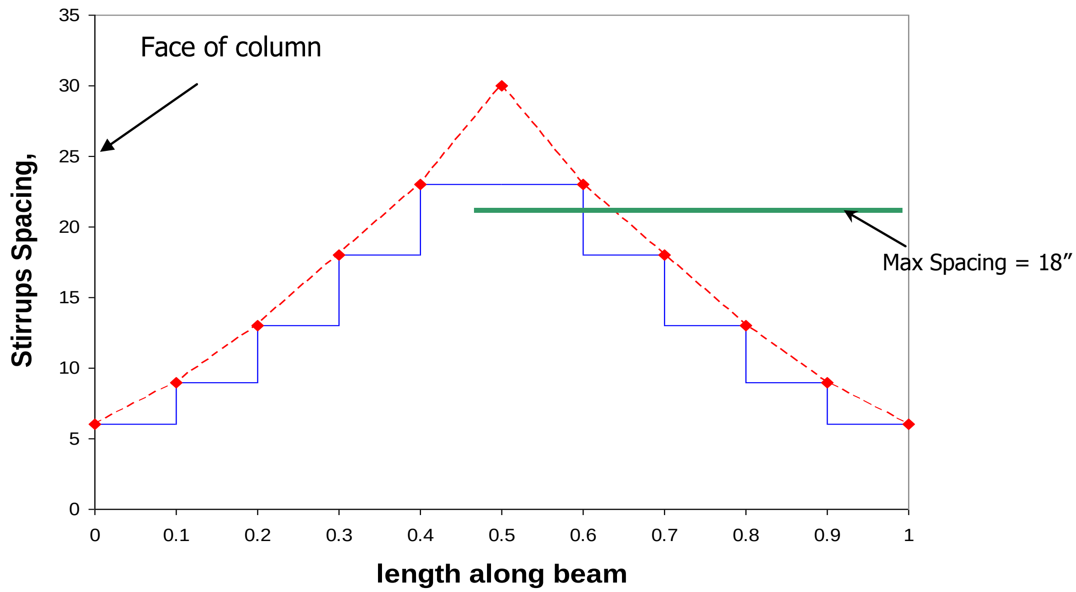


Figure 12.5-17 Stirrup Spacing along Bent Cap Length

12.5.1.10.6 Check Longitudinal Reinforcement (Shear-Flexural Interaction)

For a bent cap without prestressing steel and axial force, the longitudinal steel must satisfy:

$$A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + \left(\left| \frac{V_u}{\phi_v} \right| - 0.5 V_s \right) \cot\theta \qquad (12.3.3\text{-}1)$$

Note: In practice, after the design is complete, engineer must check seismic design requirements as per SDC and check the longitudinal steel in the bent cap to handle seismic moments.

To determine the tension in the longitudinal steel (the shear-flexure interaction), check at every location, usually at the 10th point of the span and at concentrated load:

- Maximum shear and associated moments
- Maximum positive moments and associated shear
- Maximum negative moments and associated shear

Longitudinal reinforcement at the first interior girder locations is checked in this example.

Table 12.5-5 lists unfactored shears including impact from CSiBridge analysis at the location of the first interior girder:

Table 12.5-5 Unfactored Shears at Location of First Interior Girder

	Max. Shear (kip)	Assoc. Moment (kip-ft)	Max. Moment (kip-ft)	Assoc. Shear (kip)
DC	-878	-373	-373	-878
DW	-98	-64	-64	-98
Design vehicle	-315	990	-1756	-66
Permit vehicle	-565	1,776	-3,149	-119

- Strength I

  Maximum shear and associated moments:

  V_u = 1.25(-878) + 1.5(-98) + 1.75(-315) = -1,796 kips

  $M_{u(assoc)}$ = 1.25(-373) + 1.5(-64) + 1.75(990) = 1,170 kip-ft

  Maximum moment and associated shear:

  $V_{u(assoc)}$ = 1.25(-878) + 1.5(-98) + 1.75(-66) = -1,360 kips

  $M_{u(max)}$ = 1.25(-373) + 1.5(-64) + 1.75(-1,756) = -3,635 kip-ft

- Strength II

  Maximum shear and associated moments:

  V_u = 1.25(-878) + 1.5(-98) + 1.35(-565) = $\underline{\text{-2,007 kips}}$

  $M_{u(assoc)}$ = 1.25(-373) + 1.5(-64) + 1.35(1,776) = $\underline{\text{1,835 kip-ft}}$

  Maximum moment and associate shear:

  $V_{u(assoc)}$ = 1.25(-878) + 1.5(-98) + 1.35(-119) = $\underline{\text{-1,405 kips}}$

  $M_{u(max)}$ = 1.25(-373) + 1.5(-64) + 1.35(-3,149) = $\underline{\text{-4,813 kip-ft}}$

- Check case of maximum shear and associated moment:

  $V_{u(max)}$ = -2,007 kips

  $M_{u(assoc)}$ = 1,835 kip-ft

  Compute ε_x and θ for this particular location under this loading:

  $A_{s(bot)}$ = 37.44 in.^{2}

  d_v = 73.67 – (0.85)(4.33)/2 = 71.83 in.

  $$\frac{v_u}{f'_c} = 0.0876; \ \varepsilon_x = 0.001077; \ \theta = 36.55^{\circ}$$

$$V_s = \frac{A_v f_y d_v}{s}\cot\theta = \frac{1.86(60)(71.83)}{6}\cot(36.55^o)$$

$$= 1{,}802.3 \text{ kips} < \frac{V_u}{\phi} = \frac{2{,}007}{0.9} = 2{,}230 \text{ kips}$$

$$A_s f_y \geq \frac{(1{,}835)(12)}{(0.9)(71.83)} + \left(\frac{2{,}007}{0.9} - (0.5)(1{,}663.5)\right)\cot(36.55^o)$$

$$A_s(60) \geq 340.62 + 1{,}886.18 = 2{,}226.8 \text{ kips}$$

$$A_{s(req)} = 37.11 \text{ in.}^2 < A_{s(bot)} = 37.44 \text{ in.}^2$$ OK

- Check case of maximum negative moment and associated shear

  $M_{u(max)}$ = -4,813 kip-ft
  $V_{u(assoc)}$ = -1,405 kips

  Compute ε_x and θ for this particular location under this loading:

  $A_{s(top)}$ = 28.08 in.^{2}

  d_v = 73.2 in.

  $$\frac{v_u}{f'_c} = 0.0607;\ \varepsilon_x = 0.00139;\ \theta = 36.4^o$$

  $$V_s = \frac{A_v f_y d_v}{s}\cot\theta = \frac{1.86(60)(73.2)}{6}\cot(36.4^o)$$

  $$= 1{,}846.7 \text{ kips} > \frac{V_u}{\phi} = \frac{1{,}405}{0.9} = 1{,}561 \text{ kips}$$

  Provide 22 - # 11 (A_s = 34.32 in.^{2}) as top reinforcement

- Check case of maximum positive moment and associated shear
  $M_{u(max)}$ = 1,835 kip-ft

  $V_{u(assoc)}$= -2,007 kips

  Moment and shear in this case is similar to maximum shear and associated moment case.

Note: Using the same process, one can determine the size and spacing of the stirrups at other locations.

12.5.1.11 Determine Additional Details

Additional details are discussed as follows.

12.5.1.11.1 Construction Reinforcement

As shown in Figure 12.5-18, the concrete in the superstructure is poured in two stages during the construction process. It is a Caltrans practice to assume that 10 ft of the soffit on each side of the bent cap contributes to this dead load:

- Stage 1 (first pour) includes the soffit slab and the girder stems. The deck slab is not included.
- Stage 2 (second pour) includes the deck slab.

Height of the Stage 1 pour = (6.75)(12) – 9 – 3 = 69 in. = 5.75 ft

Width of the bent cap = 8 ft

- Dead load due to exterior girder stem

  DL = [(5.75 - 8.25/12) / cos 26.6° (1)] (10 +10) (0.15) = 17 kips

- Dead load due to cap and soffit

  Width of soffit slab causing negative moment = 10 + 10 = 20 ft

  DL = {(5.75)(8) + (20)(8.25/12)}(0.15) = 9 kip/ft

  Effective column diameter = 5.32 ft

  Negative moment at the face of the column

  $M_u = 1.25\ \{[17\ (7.12 - 5.32/2)] + [9\ (7.12 - 5.32/2)^2 / 2]\ \}$

  $= 207$ kip-ft

Note: Assume a reduced value for f'_c as the concrete has not reached its specified compressive strength at the Stage 1 pour. Hence, use f'_c = 2.5 ksi. Caltrans standard practice is to provide four #10 bars as minimum construction reinforcement in the bent cap.

Since there is no prestressing steel or compression reinforcement, for simplicity of calculations, the overhangs will be neglected and nominal moment is calculated by:

$$M_n = A_s f_s \left(d_s - \frac{a}{2} \right) + 0.85 f'_c \left(b - b_w \right) h_f \left(\frac{a}{2} - \frac{h_f}{2} \right) \qquad (12.5\text{-}4)$$

b = 96 in.

d_s = 69 – 3 = 66 in.

A_s = 1.27(4) = 5.08 in.^{2}

$\beta = 0.85$

$c = (5.08)(60) / [(0.85)(2.5)(0.85)(96)] = 1.76$ in.

$a = c\,\beta = 1.494$

$M_n = 5.08\ (60)(66\text{-}1.494/2) = 19{,}889$ kip-in. = 1,657 kip-ft

$M_r = \phi M_n = 1{,}491$ kip-ft $> 1.33 M_u = 275$ kip-ft

Provide 4 # 10 ($A_s = 5.08$ in.^{2}) as construction reinforcement.

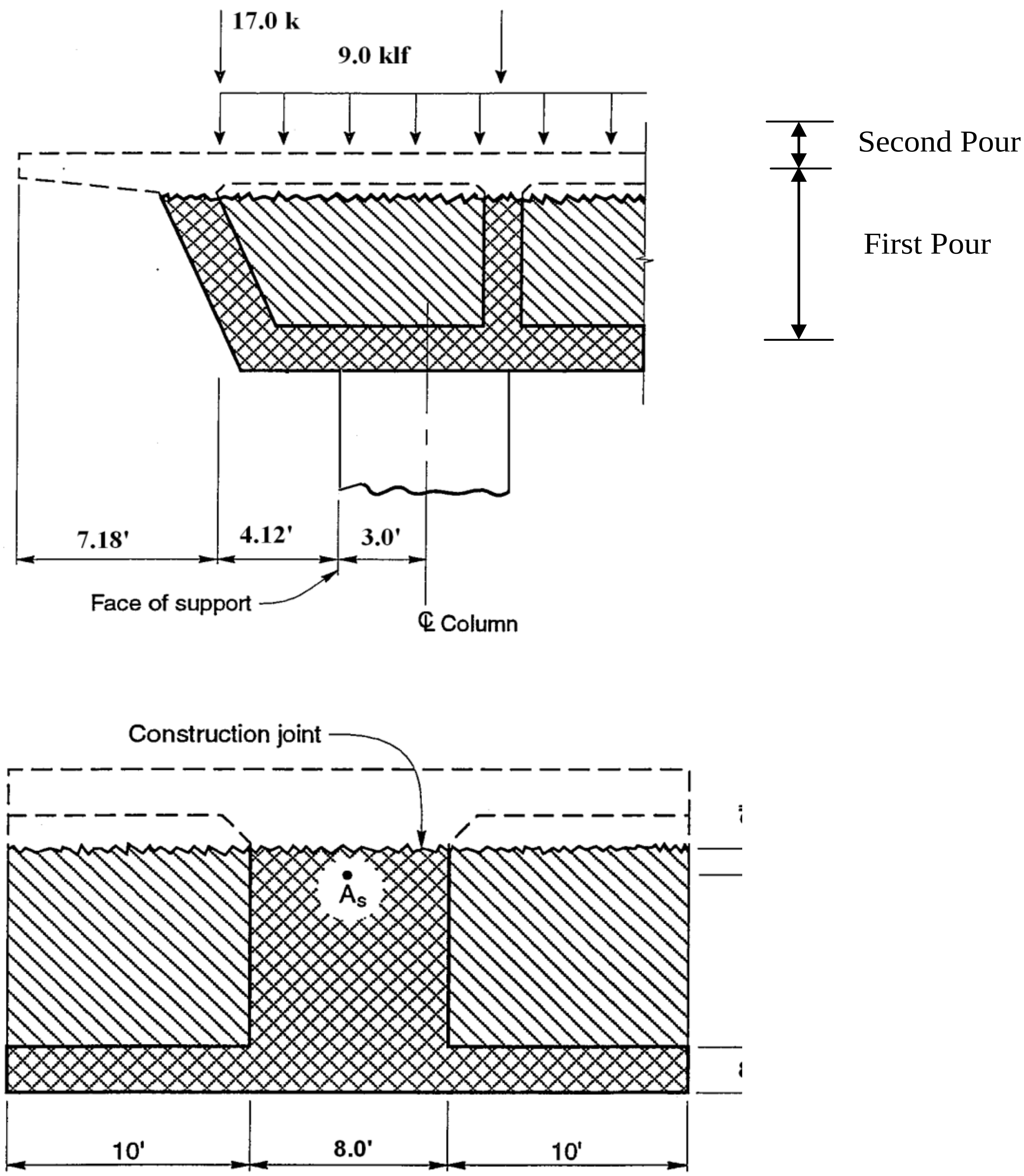


Figure 12.5-18 Concrete Pour Stages

12.5.1.11.2 Side Face / Skin Reinforcement

The Caltrans standard practice is to provide side face reinforcement, which is 10 percent of the maximum longitudinal reinforcement.

Maximum longitudinal reinforcement, $A_{s(bot)}$ = 37.44 in.^{2}

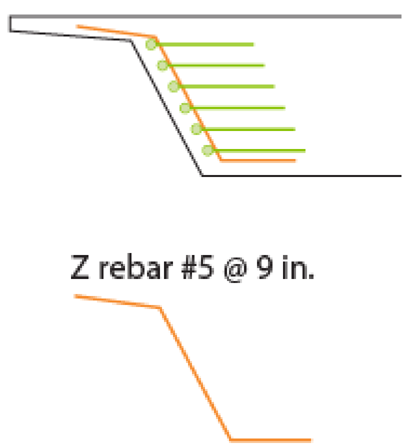
Side face reinforcement = 0.1(37.44) = 3.74 in.^{2}

Provide 10 #6 bars. Since the construction reinforcement is also provided, two #10 bars would also count as side-face reinforcement. So, provide four #6 bars on each side of the bent cap.

Spacing = {(5.75)(12) – (5.7+ 1.63) – 3} / 5 = 11.73 in. < 12 in.   OK

12.5.1.11.3 End Reinforcement

End reinforcements as shown in Figure 12.5-19 should be provided as a crack control measure, as discussed in Section 12.4.3 in accordance with BDD 7 (Caltrans, 1986).



U-bars in the Bent Cap

Figure 12.5-19 End Reinforcements

The U-bars are designed for shear friction. Depending on whether the bent cap width is less than or more than 7 ft, designer to use one or two loops of the U-bars, as shown in Figure 12.5-20. Length of U-bars shall extend a development length beyond the inside face of the exterior girder.

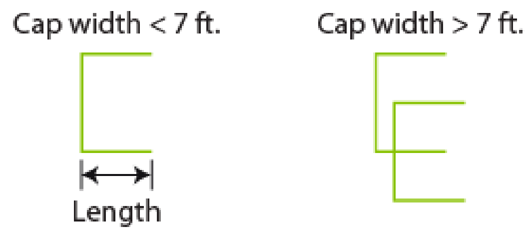


Figure 12.5-20 Length of End Reinforcements

12.5.1.11.4 Bent Cap Reinforcement

Since the bent cap skew = 20°, bent cap reinforcement shall be detailed as shown in Figure 12.5-21. A dropped deck section may be required if main cap bars are bundled vertically. Distribution bars and bottom transverse bars may have to be terminated farther from the bent cap than three in. (standard) to allow vertical clearance for main bent bars .

- Slab reinforcement parallel to skew
- Bent cap reinforcement as high as possible

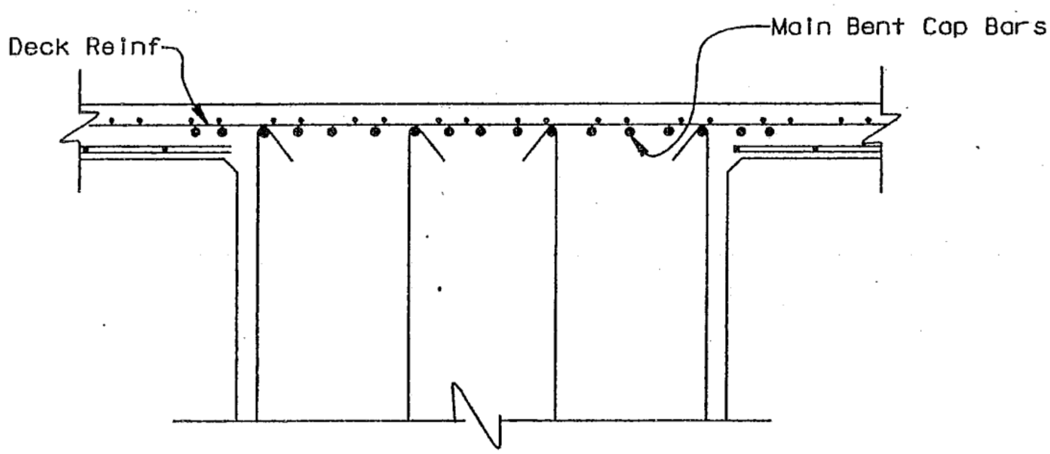


Figure 12.5-21 Bent Cap Reinforcement (Skew ≤ 20°)

2.5.2 Drop Bent Cap

A three-span bridge with reinforced concrete drop bent cap is shown in Figures 12.5-22 through 12.5-25.

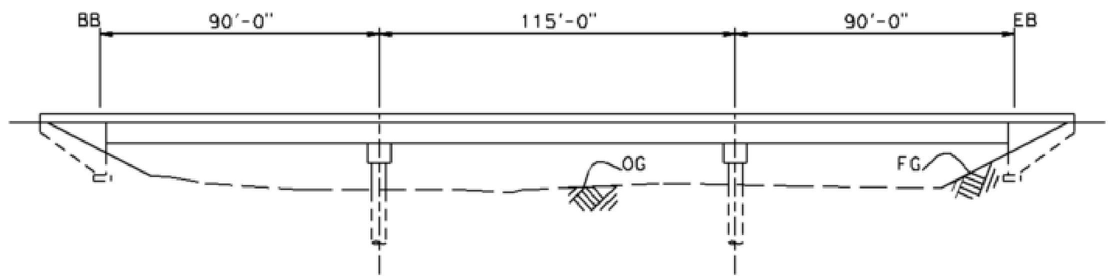


Figure 12.5-22 Elevation

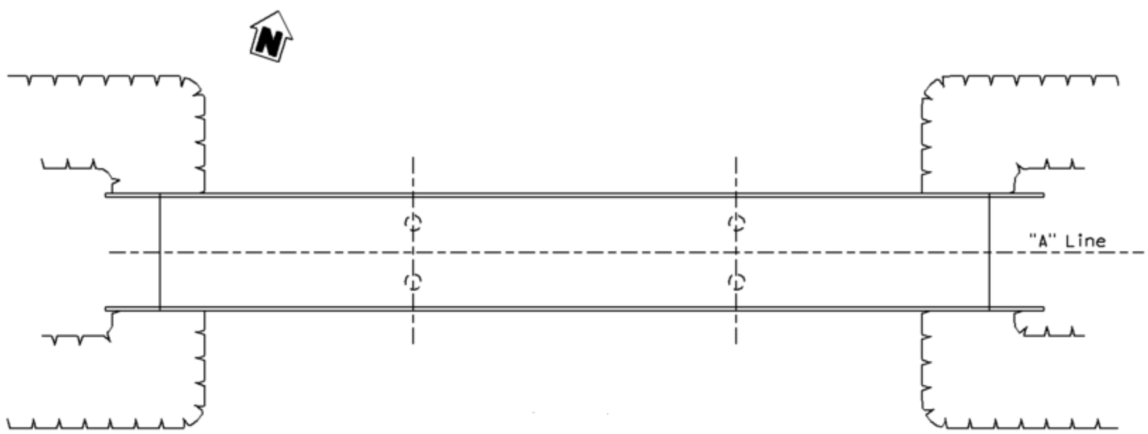


Figure 12.5-23 Plan

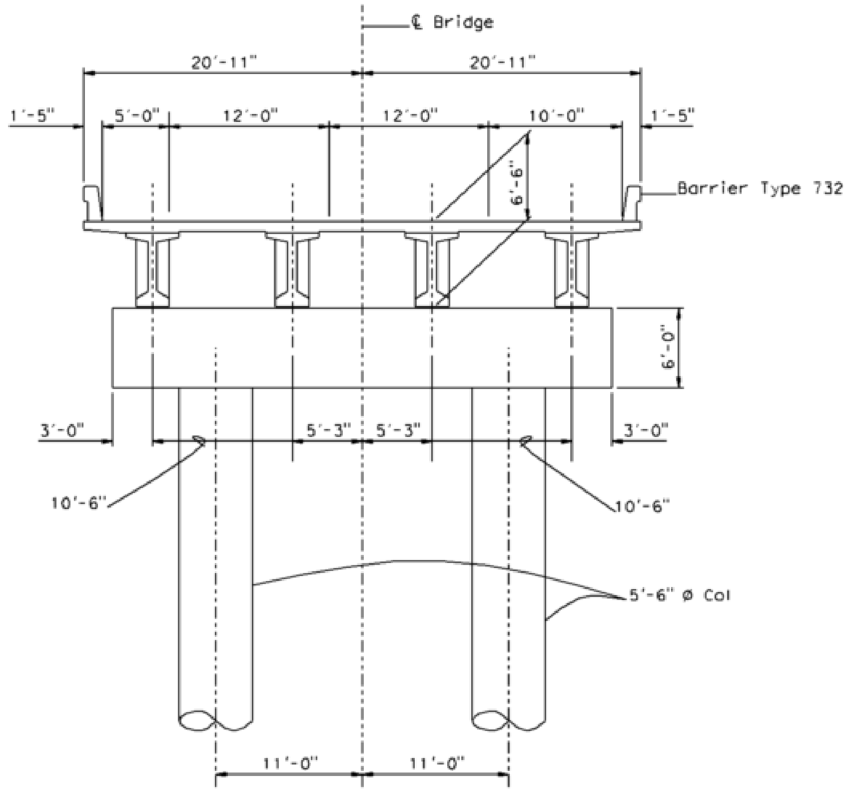


Figure 12.5-24 Typical Section

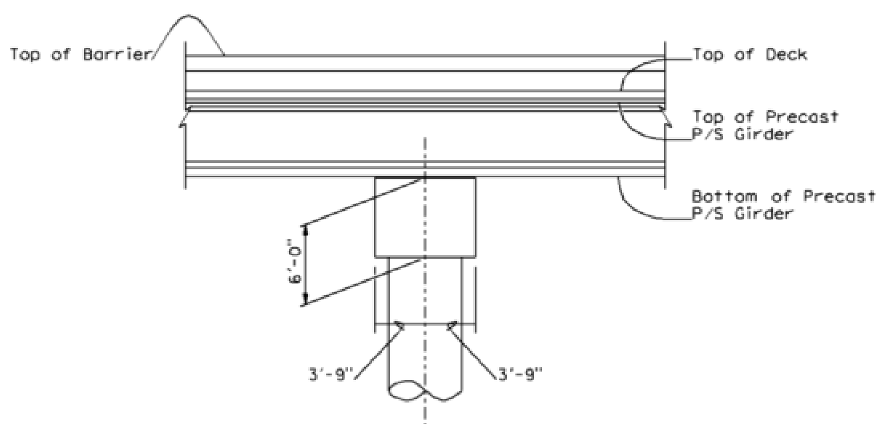


Figure 12.5-25 Side View

12.5.2.1 Member Proportioning

The bent cap depth should be deep enough to develop the column longitudinal reinforcement without hooks (SDC 7.3.4 and SDC 8.2.1). The minimum bent cap width required for adequate joint shear transfer shall be column sectional width in the direction of interest, plus two feet (SDC 7.4.2.1).

d_{cap} = 6 ft

w_{cap} = 5.5 + 2 = 7.5 ft

12.5.2.2 Classification of Bent Cap (AASHTO 5.8.1.1)

Bent caps may fall under the classification of flexural beam or deep beam. The classification dictates the type of analytical theory that would most accurately estimate the internal forces of the bent cap.

If either of these two cases is satisfied, then the bent cap may be considered a deep beam:

$L_{v\_zero} < 2d_e$

If any girder produces more than half the bent cap shear at the support and is located less than $2d_e$ from the face of support, follow this:

L_{v_zero} = 8.25 ft

$d_e = d_{cap} - 2.5$ = 69.5 in., assume 2 in. clear and an additional 0.5 in. to the centroid of flexural bar reinforcing

$2d_e$ = 11.6 ft

Both deep beam criteria are met for this example drop cap, so it should be evaluated using the strut-and-tie method per the AASHTO LRFD design specifications.

Bent caps have typically been designed by using the sectional method which has been proved to be acceptable as historical data does not suggest design inadequacies in Caltrans. This design example will follow the sectional method for its conservativeness and ease of application. Caltrans will continue to use the sectional method until the strut-and-tie method is adopted agency-wide.

12.5.2.3 Material Properties

Material properties are as follows:

f'_c = 4 ksi

f_y = 60 ksi

E_s = 29,000 ksi

12.5.2.4 Loads

CTBridge, LEAP Bridge, or other computer analysis programs can be used to determine the dead and live loads along the length of the bridge. For the bent cap design, the analysis is performed with only a single lane and/or truck so that wheel line loads may be generated and subsequently implemented in the frame analysis of the bent cap.

Generally, dead loads such as $DC_{superstructure}$ and DW are distributed by tributary area (or width) for PC/PS I girder, steel girder, and bulb T girder bridges. DC_{bent_cap}, however, is distributed along the length of the bent cap as a tributary load. In very stiff superstructures, such as cast-in-place prestressed concrete box girders, $DC_{superstructure}$ and DW may be distributed equally despite varying girder spacing. For this drop cap example, the deck, girders, bent cap, and columns will be modeled as individual elements.

Based on the longitudinal analysis, the following dead loads are applied to bent cap analytical model as shown in Figure 12.5-26.

$P_{DC\_barrier}$ = 46.5 kips

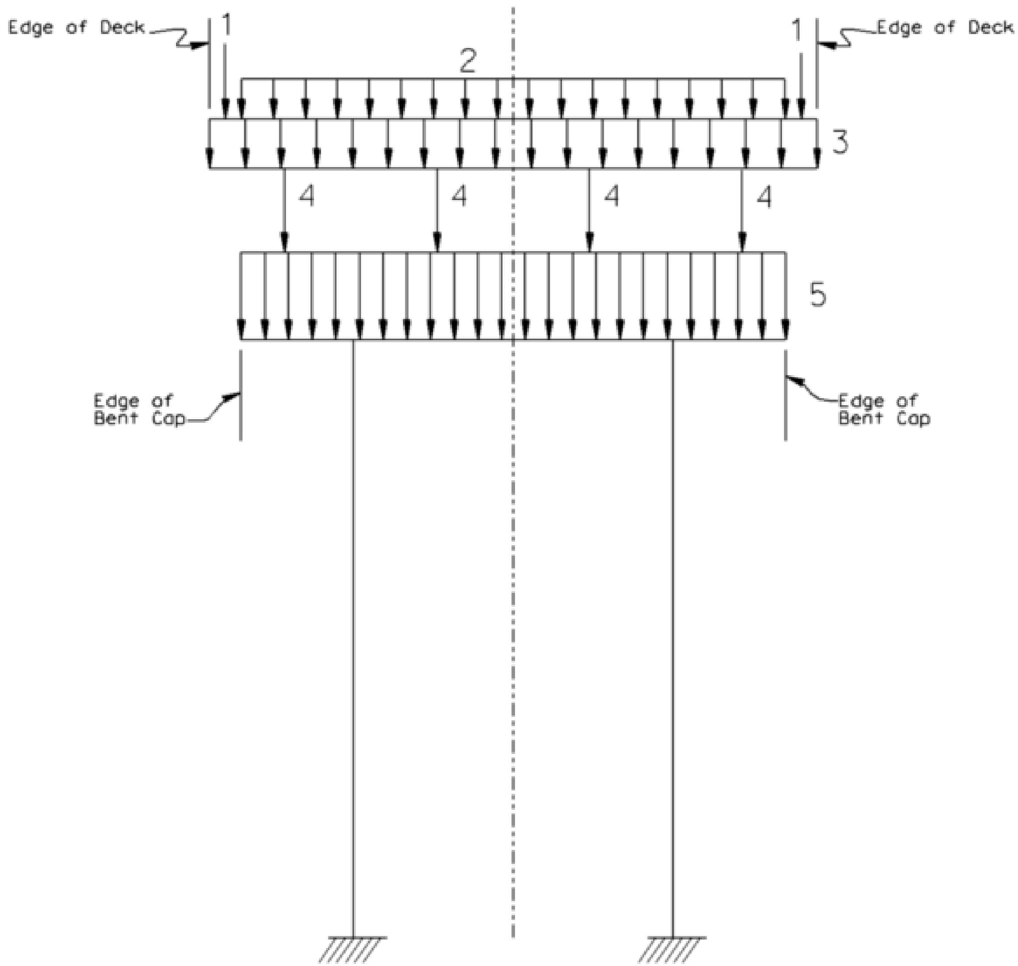
w_{DC_deck} = 12.8 kip/ft

w_{DW} = 4.4 kip/ft

P_{DC_girder} = 178 kips

$w_{DC\_bent\_cap}$ = 7.2 kip/ft

Force effects, on the bent cap, from live loads are determined similarly to the methods used for the longitudinal analysis. The live loads are discretized into wheel line loads with fixed and variable spacing to represent spacing between wheel lines, as well as spacing between trucks and lanes. Influence lines are generated to determine the governing force effects on the bent cap element. We will begin this process by generating wheel line loads from CTBridge program results.



Legend:

1 = $DC_{barrier}$ 2 = DW

3 = DC_{deck} 4 = $DC_{girders}$

5 = $DC_{bent\ cap}$

Figure 12.5-26 Dead Loads in Elevation View

12.5.2.4.1 Unfactored Bent Reactions from Longitudinal Analysis

For each design vehicular live load, the unfactored bent reactions at the bent can be obtained from the output of the longitudinal analysis. Results shown in Figures 12.5-27 to 12.5-29 are due to a single truck or lane load. The location designating "Col Bots" and "Col Tops" are the force effects to the bottoms and tops of all columns from the single truck or lane load. Results from three live loads, LRFD design vehicle (also known as HL-93), LRFD permit vehicle, and the LRFD fatigue vehicle are shown in Figures 12.5-27 to 12.5-29, respectively. The LRFD design vehicle consists of a truck and a lane. We are only interested in the maximum axial load from the truck or lane, and those values are boxed accordingly and shown as follows:

$LL_{HL\text{-}93\_truck}$	= design truck (1 lane only)	(AASHTO 3.6.1.2.2)
$LL_{HL\text{-}93\_lane}$	= lane load (1 lane only)	(AASHTO 3.6.1.2.4)
LL_{permit}	= permit vehicle (1 lane only)	(CA 3.6.1.8)
$LL_{fatigue}$	= fatigue vehicle (1 lane only)	(AASHTO 3.6.1.4)

Live Load - Controlling Unfactored Bent Reactions

Bent 2 Reactions - LRFD Design Vehicle
No Dynamic Load Allowance - Single Lane

Location	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip·ft	MZ kip·ft
Col Bots	AX-	Truck	-117.24	0.03	-0.10	2.47	1.96
		Lane	-115.19	0.05	-0.24	5.65	3.73
Col Bots	AX+	Truck	13.43	0.02	0.23	-4.97	1.27
		Lane	10.58	0.01	0.18	-3.93	1.03
Col Bots	MY-	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-47.43	-0.00	0.46	-9.70	-0.13
Col Bots	MY+	Truck	-67.31	0.07	-0.62	13.71	5.56
		Lane	-69.97	0.07	-0.54	12.05	5.31
Col Bots	MZ-	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-54.02	-0.02	0.30	-6.24	-1.26
Col Bots	MZ+	Truck	-63.92	0.07	-0.61	13.69	5.57
		Lane	-63.39	0.08	-0.38	8.59	6.43
Col Bots	VY-	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-54.02	-0.02	0.30	-6.24	-1.26
Col Bots	VY+	Truck	-63.92	0.07	-0.61	13.69	5.57
		Lane	-63.39	0.08	-0.38	8.59	6.43
Col Bots	VZ-	Truck	-67.31	0.07	-0.62	13.71	5.56
		Lane	-69.86	0.07	-0.54	12.05	5.31
Col Bots	VZ+	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-47.55	-0.00	0.46	-9.70	-0.13
Col Tops	AX-	Truck	$\boxed{-117.24}$	0.03	-0.10	-5.00	0.10
		Lane	$\boxed{-115.19}$	0.05	-0.24	-11.58	0.18
Col Tops	AX+	Truck	13.43	0.02	0.23	11.90	0.06
		Lane	10.58	0.01	0.18	9.39	0.05
Col Tops	MY-	Truck	-67.31	0.07	-0.62	-31.39	0.27
		Lane	-69.90	0.07	-0.54	-27.28	0.25
Col Tops	MY+	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-47.50	0.00	0.46	23.80	0.00
Col Tops	MZ-	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-54.02	-0.02	0.30	15.41	-0.06
Col Tops	MZ+	Truck	-63.92	0.07	-0.61	-31.24	0.27
		Lane	-63.39	0.08	-0.38	-18.88	0.31
Col Tops	VY-	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-54.02	-0.02	0.30	15.41	-0.06
Col Tops	VY+	Truck	-63.92	0.07	-0.61	-31.24	0.27
		Lane	-63.39	0.08	-0.38	-18.88	0.31
Col Tops	VZ-	Truck	-67.31	0.07	-0.62	-31.39	0.27
		Lane	-69.86	0.07	-0.54	-27.27	0.26
Col Tops	VZ+	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-47.55	-0.00	0.46	23.80	-0.01

Figure 12.5-27 LRFD Design Vehicle

Bent 2 Reactions - LRFD Permit Vehicle
No Dynamic Load Allowance - Single Lane

Location	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip·ft	MZ kip·ft
Col Bots	AX-	Truck	-368.22	0.06	-0.19	4.81	4.64
Col Bots	AX+	Truck	43.63	0.05	0.75	-16.19	4.16
Col Bots	MY-	Truck	-250.44	-0.06	1.28	-26.59	-4.38
Col Bots	MY+	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	MZ-	Truck	-250.44	-0.06	1.28	-26.59	-4.38
Col Bots	MZ+	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	VY-	Truck	-250.44	-0.06	1.28	-26.59	-4.38
Col Bots	VY+	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	VZ-	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	VZ+	Truck	-264.36	-0.06	1.28	-26.59	-4.35
Col Tops	AX-	Truck →	$\boxed{-368.22}$	0.06	-0.19	-9.27	0.23
Col Tops	AX+	Truck	43.63	0.05	0.75	38.71	0.20
Col Tops	MY-	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	MY+	Truck	-264.36	-0.06	1.28	66.83	-0.21
Col Tops	MZ-	Truck	-250.44	-0.06	1.28	66.81	-0.21
Col Tops	MZ+	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	VY-	Truck	-250.44	-0.06	1.28	66.81	-0.21
Col Tops	VY+	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	VZ-	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	VZ+	Truck	-264.36	-0.06	1.28	66.83	-0.21

Figure 12.5-28 LRFD Permit Vehicle

Bent 2 Reactions - LRFD Fatigue Vehicle
No Dynamic Load Allowance - Single Lane

Location	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip·ft	MZ kip·ft
Col Bots	AX-	Truck	-70.63	0.01	-0.03	0.66	0.50
Col Bots	AX+	Truck	9.59	0.01	0.16	-3.56	0.87
Col Bots	MY-	Truck	-52.71	-0.01	0.31	-6.42	-1.08
Col Bots	MY+	Truck	-47.45	0.05	-0.44	9.80	3.97
Col Bots	MZ-	Truck	-52.47	-0.01	0.31	-6.42	-1.08
Col Bots	MZ+	Truck	-46.71	0.05	-0.44	9.80	3.98
Col Bots	VY-	Truck	-52.47	-0.01	0.31	-6.42	-1.08
Col Bots	VY+	Truck	-46.71	0.05	-0.44	9.80	3.98
Col Bots	VZ-	Truck	-48.58	0.05	-0.44	9.80	3.97
Col Bots	VZ+	Truck	-53.09	-0.01	0.31	-6.42	-1.08
Col Tops	AX-	Truck →	$\boxed{-70.63}$	0.01	-0.03	-1.42	0.02
Col Tops	AX+	Truck	9.59	0.01	0.16	8.52	0.04
Col Tops	MY-	Truck	-48.63	0.05	-0.44	-22.44	0.19
Col Tops	MY+	Truck	-53.09	-0.01	0.31	16.13	-0.05
Col Tops	MZ-	Truck	-52.47	-0.01	0.31	16.11	-0.05
Col Tops	MZ+	Truck	-46.71	0.05	-0.44	-22.38	0.19
Col Tops	VY-	Truck	-52.47	-0.01	0.31	16.11	-0.05
Col Tops	VY+	Truck	-46.71	0.05	-0.44	-22.38	0.19
Col Tops	VZ-	Truck	-48.58	0.05	-0.44	-22.44	0.19
Col Tops	VZ+	Truck	-53.09	-0.01	0.31	16.13	-0.05

Figure 12.5-29 LRFD Fatigue Vehicle

The lane load is considered uniformly distributed over a 10-ft width. However, we will simplify the analysis by combining the HL-93 lane load with the HL-93 truck wheel line loads. Per AASHTO 3.6.1.2.4, the IM shall be applied only to the truck load. The dynamic load allowance is applied during the combination of the truck and lane load are $IM_{HL\text{-}93\_truck}$ = 1.33; and IM_{Permit} = 1.25.

$LL_{HL\text{-}93\_single\_truck}$ = 117.24 kips

$LL_{HL\text{-}93\_single\_lane}$ = 115.19 kips

$LL_{permit\_single\_truck}$ = 368.22 kips

$LL_{fatigue\_single\_truck}$ = 70.63 kips

$$LL_{HL93\_wheel\_line} = \frac{LL_{HL93\_single\_truck}\left(IM_{HL93\_truck}\right) + LL_{HL93\_single\_lane}}{2}$$

$$= \frac{117.24(1.33) + 115.19}{2} = 135.6 \text{ kips}$$

$$LL_{Permit\_wheel\_line} = \frac{LL_{Permit\_single\_truck}\left(IM_{permit}\right)}{2} = \frac{368.22(1.25)}{2} = 230.1 \text{ kips}$$

$$LL_{Fatigue\_wheel\_line} = \frac{LL_{Fatigue\_single\_truck}}{2} = \frac{70.63}{2} = 35.3 \text{ kips}$$

12.5.2.4.2 Determine Number of Live Load Lanes

Maximum number of whole live load lanes is obtained as:

Clear bridge width between curbs and/or barriers = 39 ft.

N = Integer part of (w/12) = Integer part of (39/12) = 3

12.5.2.4.3 Moving Live Load Transverse Analysis for Traffic Lanes

For this drop cap example, the design vehicle variations are shown below. This is based on the maximum number of live load lanes—a total of three—that can possibly fit in the clear roadway width. Note that the multiple presence factor, m, for one lane of permit vehicle is 1.0 (AASHTO 3.6.1.8.2), and the factor does not apply to the fatigue vehicle (AASHTO 3.6.1.1.2). Figure 12.5-30 shows traffic lanes with a multiple presence factor.

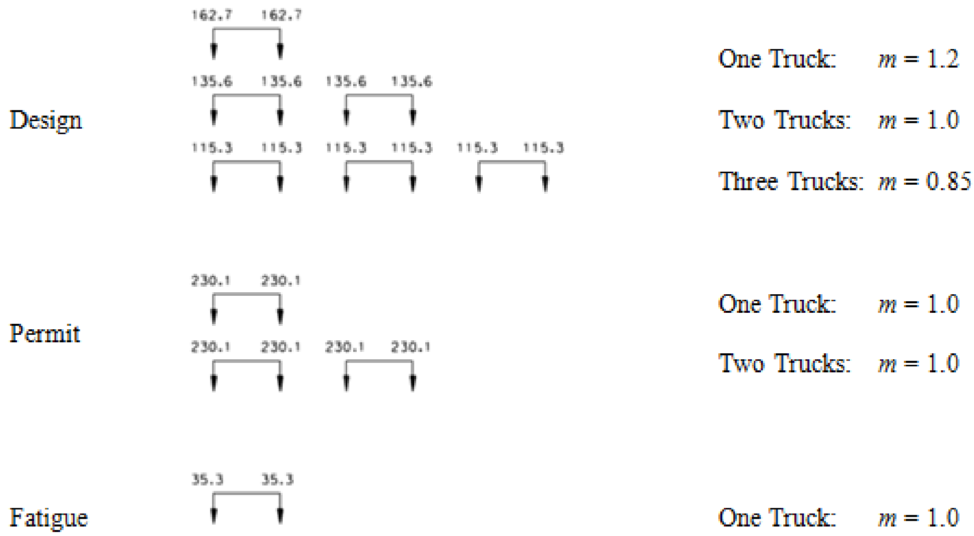


Figure 12.5-30 Types and Number of Live Loads to Apply to Bent Cap

Both the design truck and the 10 ft loaded width in each lane are positioned along the clear bridge width to produce extreme force effects. The design load is positioned transversely such that the center of any wheel load is not closer than 2 ft from the edge of the design lane (AASHTO 3.6.1.3.1).

For the moving load transverse analysis, the wheel lines may move anywhere within the 12 ft lane as long as AASHTO 3.6.1.3.1 is satisfied. Figures 12.5-31 and 32 show possible wheel line placement within the same 12-ft lane configuration. The designer must determine the placement of wheel lines that produces maximum force effects in the bent cap.

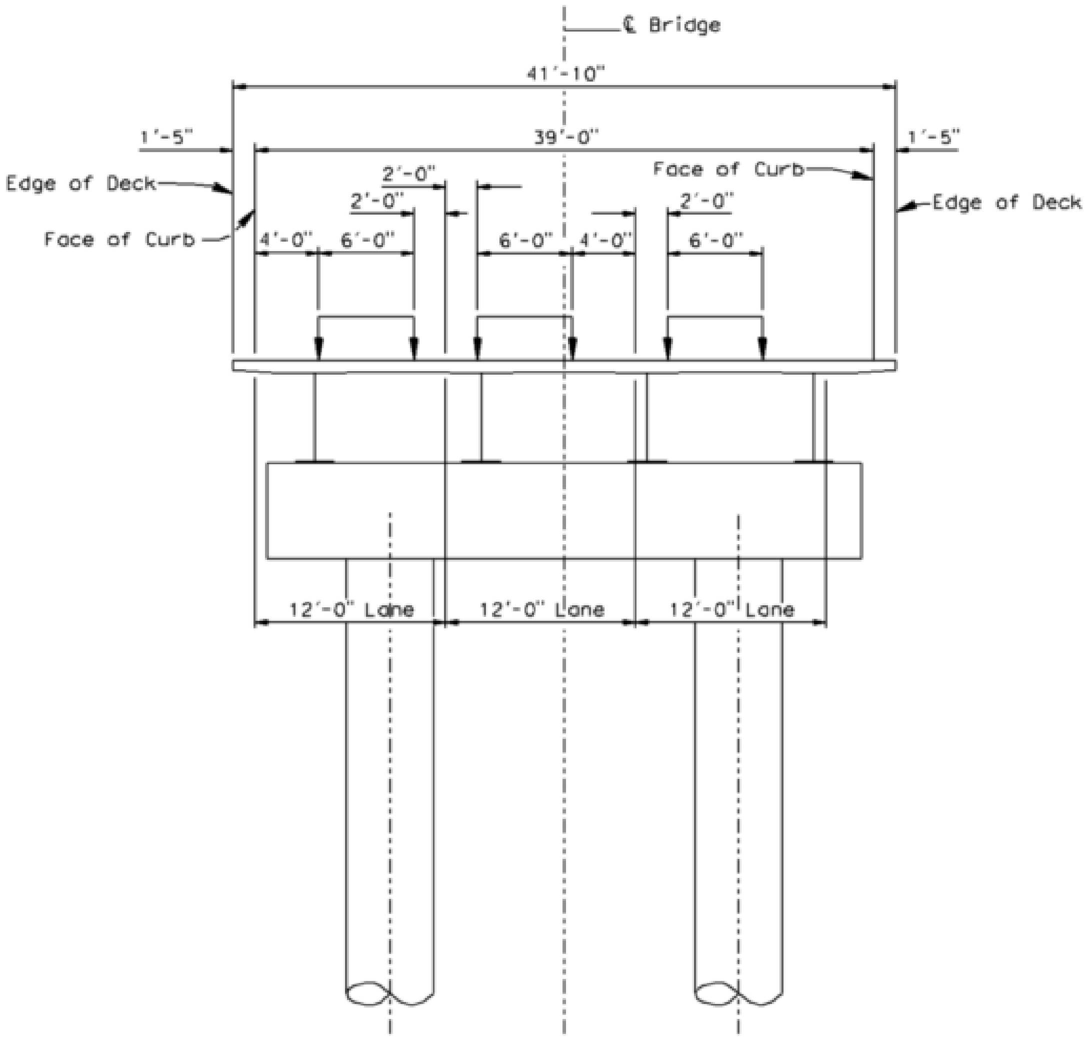


Figure 12.5-31 Example Placement of Live Load

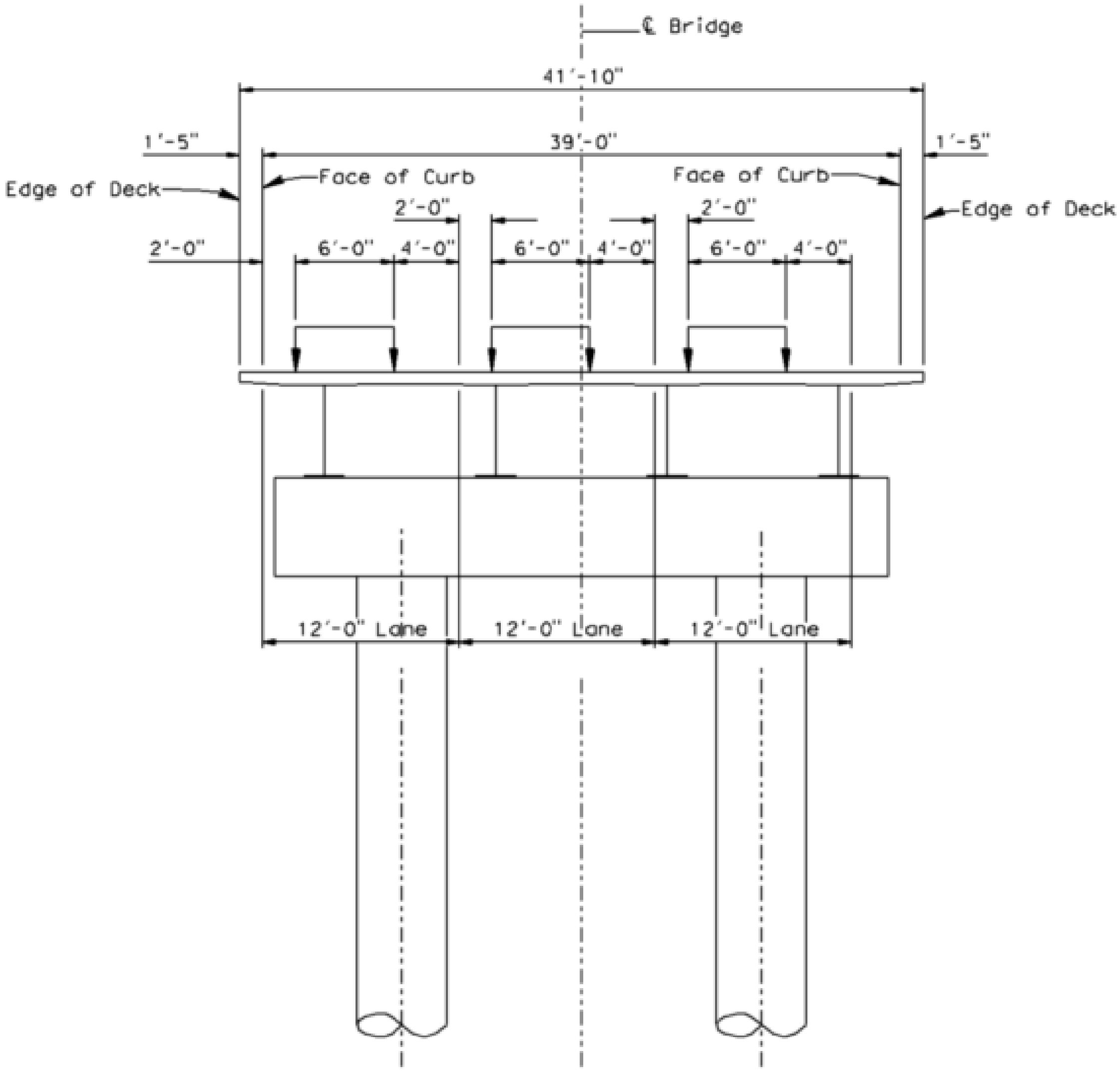


Figure 12.5-32 Another Example of Placement of Live Load

Additionally, the 12-ft lanes may move within the confines of the clear bridge width as long as no 12-ft lane overlaps another 12-ft lane. The designer must place the 12-ft lane, as well as the wheel lines, to garner the maximum force effects on the bent cap.

The designer must consider analyzing the transverse model with one, two, and three truck configurations since it is not entirely evident that three trucks will always result in the maximum force effects in the bent cap. For the cap overhang, a single vehicle placed as close to the edge of the design lane as possible may result in the maximum negative moment demand. Note that by placing only one vehicle, the multiple presence factor, *m*, is maximized and will result in a higher negative moment demand than placing two trucks with a lower multiple presence factor. For a bent cap supported by multiple columns, it is advisable to use a structural analysis program, such as CSiBridge, capable of generating combinations of lane configurations and influence lines from moving live loads. CSiBridge is used for this example.

The geometry (Figure 12.5-36) and model (Figure 12.5-37) as shown in Figures 12.5-33 and 12.5-34 consider the deck and girders atop the bent cap. Some designers may choose to construct a hybrid frame in which the deck, girders, and bent cap are represented by an integrated horizontal frame member. For the purpose of maintaining a simplified representation, we are opting to keep the members separate.

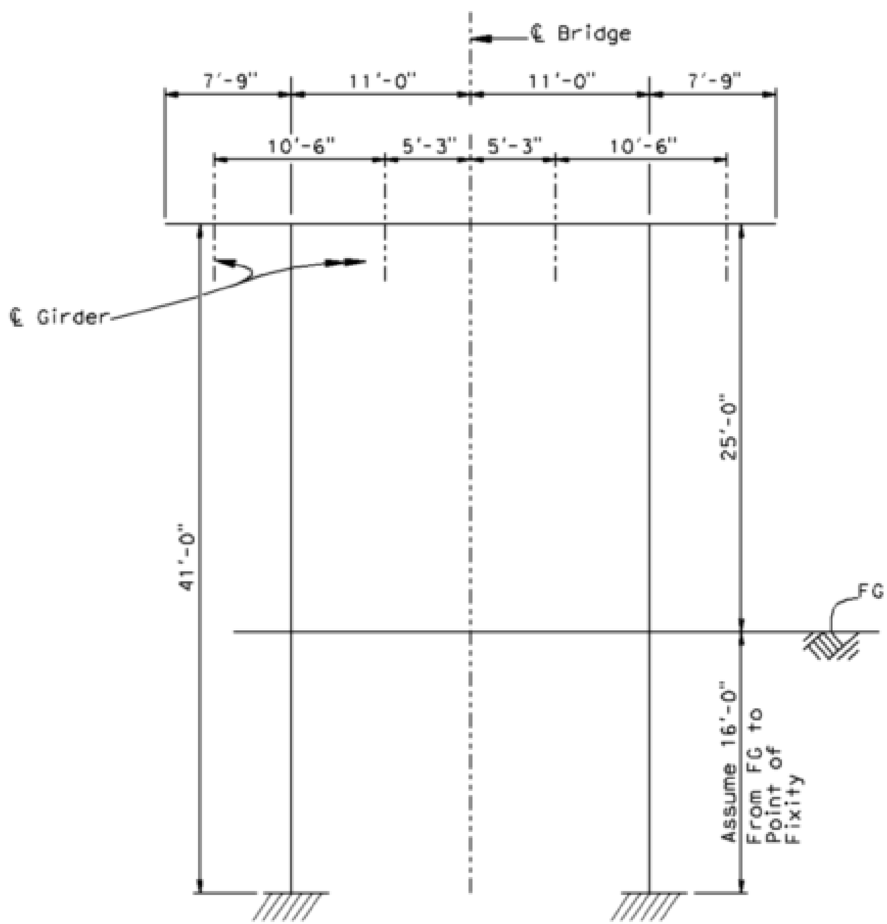


Figure 12.5-33 Geometric Model of Bent Cap

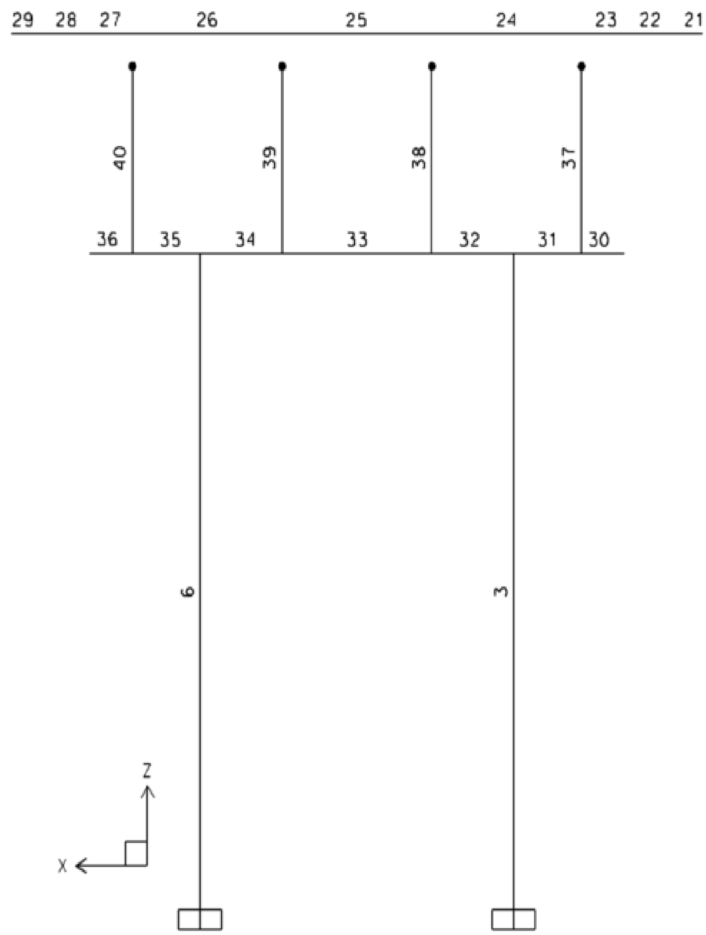


Figure 12.5-34 CSiBridge Model Showing Member Designations

For each vehicular load (HL-93, Permit, and Fatigue), the output will be organized in the following manner:

Max *M3*, associated *V2*

Min *M3*, associated *V2*

Max *V2*, associated *M3*

Min *V2*, associated *M3*

Breaking down the demands in such a manner allows us to combine them with dead loads and apply the appropriate load factors. The associated force effects are necessary for checking moment-shear interaction (AASHTO 5.8.3.5).

The bent cap locations of interest will be positive moments at midspan, negative moments at faces of column, and shear at faces of column.

12.5.2.4.4 Frame Element 33 (Max Positive Moment at Midspan)

Figures 12.5-35 to 12.5-38 are excerpts from the CSiBridge output to illustrate how moments are extracted from the analysis.

	A	B	C	D	E	F	G	H	I	J	K
1	TABLE: Element Forces - Frames										
2	Frame	Station	OutputCase	CaseType	StepType	P	V2	V3	T	M2	M3
3	Text	ft	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
286	33	2.33333333	DC	LinStatic		2.41812152	-20.9997799	0	0	0	41.75486128
287	33	3.5	DC	LinStatic		2.41812152	-12.5997799	0	0	0	61.35460446
288	33	4.66666667	DC	LinStatic		2.41812152	-4.19977987	0	0	0	71.15434764
289	33	5.83333333	DC	LinStatic		2.41812152	**4.2002201**	0	0	0	**71.154091**
290	33	7	DC	LinStatic		2.41812152	12.60022013	0	0	0	61.353834
291	33	8.16666667	DC	LinStatic		2.41812152	21.00022013	0	0	0	41.75357719
292	33	9.33333333	DC	LinStatic		2.41812152	29.40022013	0	0	0	12.35332037
293	33	10.5	DC	LinStatic		2.41812152	37.80022013	0	0	0	-26.8469365
294	33	0	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.82079626
295	33	1.16666667	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.82078897
296	33	2.33333333	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.82078167
297	33	3.5	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.82077438
298	33	4.66666667	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.82076709
299	33	5.83333333	DW	LinStatic		-0.53613537	**6.25E-06**	0	0	0	**73.82076**
300	33	7	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.8207525
301	33	8.16666667	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.8207452
302	33	9.33333333	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.82073791
303	33	10.5	DW	LinStatic		-0.53613537	6.25E-06	0	0	0	73.82073062
304	33	0	HL93	LinMoving	Max V2	-0.57561569	97.96574813	0	0	0	627.8846823
305	33	1.16666667	HL93	LinMoving	Max V2	-0.57561569	97.96574813	0	0	0	513.5913095
306	33	2.33333333	HL93	LinMoving	Max V2	-0.57561569	97.96574813	0	0	0	399.2979367
307	33	3.5	HL93	LinMoving	Max V2	-0.57561569	97.96574813	0	0	0	285.0045638

Figure 12.5-35 Positive Moment from Dead Loads (*DC* and *DW*) at Midspan of Cap

	A	B	C	D	E	F	G	H	I	J	K
1	TABLE: Element Forces - Frames										
2	Frame	Station	OutputCase	CaseType	StepType	P	V2	V3	T	M2	M3
3	Text	ft	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
322	33	9.33333333	HL93	LinMoving	Min V2	-0.57561778	-97.9657182	0	0	0	513.591372
323	33	10.5	HL93	LinMoving	Min V2	-0.57561778	-97.9657182	0	0	0	627.8847099
324	33	0	HL93	LinMoving	Max M3	-11.3525008	18.14510731	0	0	0	1181.895807
325	33	1.16666667	HL93	LinMoving	Max M3	-11.3525008	14.11286124	0	0	0	1181.895807
326	33	2.33333333	HL93	LinMoving	Max M3	-11.3525008	10.08061517	0	0	0	1181.895807
327	33	3.5	HL93	LinMoving	Max M3	-11.3525008	6.048369103	0	0	0	1181.895807
328	33	4.66666667	HL93	LinMoving	Max M3	-11.3525008	**2.016123**	0	0	0	**1181.8958**
329	33	5.83333333	HL93	LinMoving	Max M3	-11.3525008	-2.01612303	0	0	0	1181.895807
330	33	7	HL93	LinMoving	Max M3	-11.3525008	-6.0483691	0	0	0	1181.895807
331	33	8.16666667	HL93	LinMoving	Max M3	-11.3525008	-10.0806152	0	0	0	1181.895807
332	33	9.33333333	HL93	LinMoving	Max M3	-11.3525008	-14.1128612	0	0	0	1181.895807
333	33	10.5	HL93	LinMoving	Max M3	-11.3525008	-18.1451073	0	0	0	1181.895807
334	33	0	HL93	LinMoving	Min M3	8.365206393	-12.8790747	0	0	0	-743.996543
335	33	1.16666667	HL93	LinMoving	Min M3	8.365206178	-10.0170487	0	0	0	-743.996573

Figure 12.5-36 Maximum Positive Moment from HL-93 at Midspan of Cap

	A	B	C	D	E	F	G	H	I	J	K
1	TABLE: Element Forces - Frames										
2	Frame	Station	OutputCase	CaseType	StepType	P	V2	V3	T	M2	M3
3	Text	ft	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
364	33	0	Permit	LinMoving	Max M3	-19.2640887	30.79048076	0	0	0	2005.562133
365	33	1.16666667	Permit	LinMoving	Max M3	-19.2640887	23.94815171	0	0	0	2005.562133
366	33	2.33333333	Permit	LinMoving	Max M3	-19.2640887	17.10582265	0	0	0	2005.562133
367	33	3.5	Permit	LinMoving	Max M3	-19.2640887	10.26349359	0	0	0	2005.562133
368	33	4.66666667	Permit	LinMoving	Max M3	-19.2640887	**3.4211645**	0	0	0	**2005.5621**
369	33	5.83333333	Permit	LinMoving	Max M3	-19.2640887	-3.42116453	0	0	0	2005.562133
370	33	7	Permit	LinMoving	Max M3	-19.2640887	-10.2634936	0	0	0	2005.562133
371	33	8.16666667	Permit	LinMoving	Max M3	-19.2640887	-17.1058226	0	0	0	2005.562133
372	33	9.33333333	Permit	LinMoving	Max M3	-19.2640887	-23.9481517	0	0	0	2005.562133
373	33	10.5	Permit	LinMoving	Max M3	-19.2640887	-30.7904808	0	0	0	2005.562133
374	33	0	Permit	LinMoving	Min M3	14.19494094	-21.854536	0	0	0	-1262.48971
375	33	1.16666667	Permit	LinMoving	Min M3	14.19494057	-16.9979566	0	0	0	-1262.48976

Figure 12.5-37 Max Positive Moment from Permit Truck at Midspan of Cap

	A	B	C	D	E	F	G	H	I	J	K
1	TABLE: Element Forces - Frames										
2	Frame	Station	OutputCase	CaseType	StepType	P	V2	V3	T	M2	M3
3	Text	ft	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
403	33	10.5	Fatigue	LinMoving	Min V2	-1.23186337	-13.9997177	0	0	0	193.2198202
404	33	0	Fatigue	LinMoving	Max M3	-1.74890202	10.6583931	0	0	0	221.3596352
405	33	1.16666667	Fatigue	LinMoving	Max M3	-1.74890202	8.2898613	0	0	0	221.3596352
406	33	2.33333333	Fatigue	LinMoving	Max M3	-1.74890202	5.9213295	0	0	0	221.3596352
407	33	3.5	Fatigue	LinMoving	Max M3	-1.74890202	3.5527977	0	0	0	221.3596352
408	33	4.66666667	Fatigue	LinMoving	Max M3	-1.74890202	**1.1842659**	0	0	0	**221.35964**
409	33	5.83333333	Fatigue	LinMoving	Max M3	-1.74890202	-1.1842659	0	0	0	221.3596352
410	33	7	Fatigue	LinMoving	Max M3	-1.74890202	-3.5527977	0	0	0	221.3596352
411	33	8.16666667	Fatigue	LinMoving	Max M3	-1.74890202	-5.9213295	0	0	0	221.3596352
412	33	9.33333333	Fatigue	LinMoving	Max M3	-1.74890202	-8.2898613	0	0	0	221.3596352
413	33	10.5	Fatigue	LinMoving	Max M3	-1.74890202	-10.6583931	0	0	0	221.3596352
414	33	0	Fatigue	LinMoving	Min M3	1.082015963	-11.5031573	0	0	0	-150.549209
415	33	1.16666667	Fatigue	LinMoving	Min M3	1.082016023	-8.94689927	0	0	0	-150.549219

Figure 12.5-38 Max Positive Moment from Fatigue Truck at Midspan of Cap

Maximum positive moments for Frame Element 33 obtained from CSiBridge output are summarized in Tables 12.5-6.

Table 12.5-6 Maximum Positive Moments for Frame Element 33

Dead Loads	Live Load ($M3$)	Live Load (Associated $V2$)
M_{DC_33} = 71.2 kip-ft	$M_{HL\text{-}93\_33}$ = 1181.9 kip-ft	$V_{HL\text{-}93\_33}$ = 2 kips
V_{DC_33} = 4.2 kips	M_{Permit_33} = 2005.6 kip-ft	V_{Permit_33} = 3.4 kips
V_{DW_33} = 73.8 kip-ft	M_{DC_33} = 71.2 kip-ft	M_{DC_33} = 71.2 kip-ft
M_{DC_33} = 71.2 kip-ft		

This example calculation will evaluate the bent cap at the right face of the left column, also referred to as the left end of Frame Element 34. Maximum negative moments for Frame Element 34 obtained from CSiBridge are summarized in Table 12.5-7.

Table 12.5-7 Maximum Negative Moment for Frame Element 34

Dead Loads	Live Load ($M3$)
M_{DC_34} = -1127.5 kip-ft	$M_{HL\text{-}93\_34}$ = -904.9 kip-ft
V_{DC_34} = 344.5 kips	M_{Permit_34} = -1530.7 kip-ft
M_{DW_34} = -82.6 kip-ft	$M_{Fatigue\_34}$ = -196.3 kip-ft
V_{DW_34} = 71.2 kip-ft	

12.5.2.5 Design for Flexure

The software WinConc are used to design for flexure in this example. Note that in WinConc, there are optional inputs for other loads discussed thoroughly in AASHTO Article 3.6, 3.7, 3.8, 3.9, 3.10, 3.11, 3.12, 3.13, and 3.14. Of those loads, CR, SH, wind load on live load (WL), wind load on structure (WS), temperature loads (TU), and differential settlement (SE) can impose force effects in the bent cap.

By virtue of experience, flexural and shear design is almost always governed by the Strength I and Strength II limit states. Since WL and WS are not considered in either of these load combinations (AASHTO Table 3.4.1-1), force effects from wind are not computed. However, uniform temperature is considered in both Strength I and Strength II limit states, but because of the relatively short distance between the two columns, it is anticipated that the force effects generated by TU is insignificant.

Differential shrinkage pertains to strains generated between material of different age or composition. This example drop cap will be built monolithically; therefore, SH is not considered. Creep is a force effect that is generated by prestressed concrete elements. This example drop cap will be conventionally reinforced with mild reinforcement; therefore CR is also not considered.

For this example, differential settlement will not be considered. Generally, the geotechnical engineer dictates the consideration of differential settlement. SE may impose force effects in the bent cap if the soil profiles between the two columns differ, thereby causing one column to settle more than the other column.

Two WinConc models were assembled: one for the design of the bottom flexure reinforcing (positive moment) and the other for the design of the top flexure reinforcing (negative moment). WinConc allows the user to enter the bent cap dimensions and loads. Then, it tabulates the rebar size and quantity that satisfies the AASHTO LRFD design specifications. Figures 12.5-39 and 12.5-40 are excerpts from the output files to show how the results can be used

```
**************************
* Final Results - DESIGN *
*************************************************************************

(Stress Analysis and Area of Steel Required for Different Bar Sizes)

             :---------Service---------:---Fatigue---:
                           Actual  Allow Actual  Allow
Bar  Load    Actual Allow    Bar    Bar Stress Stress    Eff     Req Total Space
Size Control Stress Stress Space  Space  Range  Range  Depth   Steel  Bars  Code
              (Ksi)  (Ksi)  (in)   (in)  (Ksi)   (Ksi)  (in)  (in^2)   Req
---- ------- ------ ------ ------ ------ ------ ------ ------ ------- ----- -----
 18  S-As      15.3          29.9   36.5    4.5   22.9   68.8    14.8    4   3
 14  Min2      17.9          18.1   30.9    5.2   22.8   69.1    12.6    6   3
 11  Min2      17.2          11.3   32.8    5.0   22.8   69.2    12.6    9
 10  Min2      19.0          10.1   29.5    5.5   22.7   69.3    12.6   10
  9  Min2      18.5           7.6   30.6    5.4   22.7   69.4    12.5   13
  8  Min2      19.0           6.1   29.8    5.6   22.7   69.4    12.5   16
  7  Min2      19.1           4.6   29.9    5.6   22.7   69.5    12.5   21
  6  Min2      18.8           3.3   30.6    5.5   22.7   69.6    12.5   29   2
  5  Min2      18.9           2.3   30.7    5.5   22.7   69.7    12.5   41   1

Space Codes

1 = Bar Spacing Less Than The AASHTO Minimum (8.21)
2 = Bar Spacing Less Than The Preferred CALTRANS Minimum. (Br. Des. Det.)
3 = Bar Spacing More Than THE AASHTO Maximum (18" or 457.2 mm) (Code 8.20)
```

Figure 12.5-39 Partial WinConc Output for Positive Flexure Design

```
**************************
* Final Results - DESIGN *
*************************************************************************

(Stress Analysis and Area of Steel Required for Different Bar Sizes)

             :---------Service---------:---Fatigue---:
                           Actual  Allow Actual  Allow
Bar  Load    Actual Allow    Bar    Bar Stress Stress    Eff     Req Total Space
Size Control Stress Stress Space  Space  Range  Range  Depth   Steel  Bars  Code
              (Ksi)  (Ksi)  (in)   (in)  (Ksi)   (Ksi)  (in)  (in^2)   Req
---- ------- ------ ------ ------ ------ ------ ------ ------ ------- ----- -----
 18  S-As      19.6          22.4   26.9    3.2   19.7   68.8    18.6    5   3
 14  Min2      24.7          15.1   20.9    4.0   18.7   69.1    15.7    7
 11  Min2      22.6           9.1   23.6    3.7   19.1   69.2    15.7   11
 10  Min2      23.5           7.6   22.8    3.8   19.0   69.3    15.6   13
  9  Min2      24.2           6.1   22.2    3.9   18.8   69.4    15.6   16
  8  Min2      24.5           4.8   22.1    4.0   18.8   69.4    15.6   20
  7  Min2      24.7           3.6   21.9    4.0   18.7   69.5    15.6   26   2
  6  Min2      24.3           2.6   22.5    4.0   18.8   69.6    15.6   36   1

Space Codes

1 = Bar Spacing Less Than The AASHTO Minimum (8.21)
2 = Bar Spacing Less Than The Preferred CALTRANS Minimum. (Br. Des. Det.)
3 = Bar Spacing More Than THE AASHTO Maximum (18" or 457.2 mm) (Code 8.20)
```

Figure 12.5-40 Partial WinConc Output for Negative Flexure Design

When WinConc is run in design mode, the "final results" show the various bar size and spacing configurations that satisfy the strength, service, fatigue, and extreme limit states, as well as crack control (AASHTO 5.7.3.4) and minimum reinforcement (AASHTO 5.7.3.3.2) requirements. It is worth noting that any bar configuration flagged with "space code" is undesirable because of substandard bar spacing.

The following "load controls" are checked, and the governing "load control" is summarized in the final results:

S-As = Service I, bar spacing for crack control (AASHTO 5.7.3.4)

F-As = Fatigue I, fatigue stress in mild steel (AASHTO 5.5.3.2)

Str-I = HL-93 loads

Str-II = Permit loads

Str-III = No HL-93 loads, wind > 55 mph

Str-IV = No HL-93 loads, governs when DL to LL ratio is high

Str-V = HL-93 loads, wind = 55 mph

Ext-I = Earthquake

Ext-II = Ice, train, vehicle, or vessel collision

Arb-I = User defined load

Min1 = Minimum reinforcement requirement, M_{cr}

Min2 = Waiver of minimum reinforcement requirement, 1.33 M_u

For flexural design in positive bending, Min2 governed for all bar sizes except the #18 bar. Of the available bars, #7, #8, #9, #10, and #11 provide acceptable capacity while satisfying bar spacing requirements. A total of 16 #8 bars are used.

For flexural design in negative bending, Min2 also governed for all bar sizes except the #18 bar. Of the available bars, #8, #9, #10, #11, and #14 provide acceptable capacity while satisfying bar spacing requirements. We'll specify a total of 16 #10 bars, primarily so that they lineup with the 16 #8 main bottom bars. Although 16 #9 bars would suffice, some engineers believe it is good practice to specify bars sizes that are not too similar in size when bars can potentially be mixed up during construction.

In summary, the bar reinforcement areas corresponding to positive and negative moments are as such:

$A_{s\ positive}$ = 16 (0.79 in.^{2}) = 12.6 in.^{2}

$A_{s\ negative}$ = 16 (1.27 in.^{2}) = 20.3 in.^{2}

12.5.2.6 Design for Shear

The AASHTO LRFD shear design method is based on the modified compression field theory. Contrary to the traditional shear design methodology, it assumes a variable angle truss model instead of the 45° truss analogy. The LRFD method accounts for interaction between shear, torsion, flexure, and axial load.

The LRFD method notes that shear design will be considered to the distance, d_v, from the face of support (AASHTO 5.8.3.2). However, Caltrans' practice is to evaluate shear to the face of support.

12.5.2.6.1 Calculate Factored Shear

Unfactored shears and associated moments for the left face of the left column, Frame Element 34, extracted from CSiBridge output are summarized in Table 12.5-8.

Table 12.5-8 Maximum Shears and Associated Moments for Frame Element 34 (Interior Face of Column)

Shear Demand (*V*2)	**Associated Moments (*M*3)**
V_{DC_34} = 344.5 kip	M_{DC_34} = -1127.5 kip-ft
V_{DW_34} = 47.3 kip	M_{DW_34} = -82.6 kip-ft
$V_{HL\text{-}93\_34}$ = 300.6 kip	$M_{HL\text{-}93\_34}$ = -196.3 kip-ft
V_{Permit_34} = 510.1 kip	M_{Permit_34} = 114.8 kip-ft

Two load combinations, Strength I and II typically govern for shear design of bent caps. Factored shear demand and associated factored moments for frame element 34 are calculated as:

$$V_{u\_34\_Strength\_1} = 1.25\,(V_{DC\_34}) + 1.5\,(V_{DW\_34}) + 1.75\,(V_{HL\text{-}93\_34}) = 1{,}027.6 \text{ kips}$$

$$M_{u\_34\_Strength\_1} = 1.25\,(M_{DC\_34}) + 1.5\,(M_{DW\_34}) + 1.75\,(M_{HL\text{-}93\_34}) = -1{,}876.8 \text{ kip-ft}$$

$$V_{u\_34\_Strength\_I1} = 1.25\,(V_{DC\_34}) + 1.5\,(V_{DW\_34}) + 1.35\,(V_{Permit\_34}) = 1{,}190.2 \text{ kips}$$

$$M_{u\_34\_Strength\_I1} = 1.25\,(M_{DC\_34}) + 1.5\,(M_{DW\_34}) + 1.35\,(M_{Permit\_34}) = -1{,}378.3 \text{ kip-ft}$$

$$V_{u\_34} = V_{u\_34\_Strength\_I1} = 1{,}190.2 \text{ kips} \qquad \leftarrow \text{Strength II governs}$$

$$M_{u\_34} = M_{u\_34\_Strength\_I1} = -1378.3 \text{ kip-ft}$$

12.5.2.6.2 Determine θ and β

Cross section of the drop cap is shown in Figure 12.5-41. b_v is width of web taken as 96 in; d_e is effective depth taken as 69.5 in; d_v is effective shear depth taken as the max (d_{v1}, 0.9 d_e, 0.72d_{cap}); d_{cap} is depth of cap taken as 72 in.

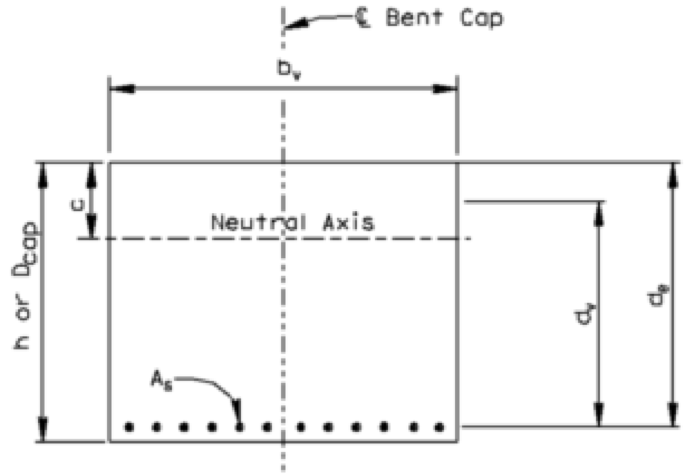


Figure 12.5-41 Cross Section of Drop Cap

Assuming $f_s = f_y$, we have

$$a = \frac{\left(A_{s\ negaitve} f_y - A_{s\ positive} f_y\right)}{0.85 f_c' b_v} = \frac{(20.3)(60) - (12.6)(60)}{(0.85)(4)(96)} = 1.42 \text{ in.}$$

$$d_{v1} = d_e - \frac{a}{2} = 68.8 \text{ in.}$$

d_v = max (d_{v1}, 0.9 (d_e), 0.72 (d_{cap}) = 68.8 in.

$$v_u = \frac{V_{u34}}{\phi_v b_v d_v} = \frac{1{,}190.2}{(0.9)(96)(68.8)} = 0.2 \text{ ksi} \qquad \text{(AASHTO 5.8.2.9-1)}$$

Shear stress factor:

$$v_u = \frac{v_u}{f_c'} = \frac{0.2}{4} = 0.05$$

Assume (0.5cot θ = 1) and use absolute values for M_u and V_u for strain calculation, we have:

$$\varepsilon_x = \frac{\dfrac{|M_{u34}|}{d_v} + 0.5V_{u34}(\cot\theta)}{2E_s A_{s\ negative}} = \frac{\left(\dfrac{(1{,}378.3)(12)}{68.6} + 1{,}190.2\right)}{2(29{,}000)(20.3)} = 0.00121$$

From AASHTO Table B5.2-1, β = 2.23 and θ = 36.4° are obtained.

Use θ = 36.4° and recalculate ε_x as follows:

$$\varepsilon_x = \frac{\dfrac{\left|M_{u34}\right|}{d_v} + 0.5V_{u34}(\cot\theta)}{2E_s A_{s\ negative}} = \frac{\left(\dfrac{(1{,}378.3)(12)}{68.6} + (0.5)(1{,}190.2)(\cot 36.4^o)\right)}{2(29{,}000)(20.3)} = 0.00089$$

Form AASHTO Table B5.2-1, β = 2.23 and θ = 36.4° are obtained again. And convergence is reached.

12.5.2.6.3 Determine Shear Reinforcement

Concrete contribution to shear resistance:

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v = (0.0316)(2.23)\sqrt{4}(96)(68.6) = 928.1 \text{ kips}$$

(AASHTO 5.8.3.3)

$$V_s = \frac{V_{u34}}{\phi_v} - V_c = \frac{1{,}190.2}{0.9} - 928.1 = 394.3 \text{ kips}$$

Required shear stirrups:

$$\frac{A_v}{s} = \frac{V_s}{f_y d_v \cot\theta} = \frac{1{,}190.2}{(60)(68.6)(\cot 36.4^o)} = 0.212 \text{ in.}^2/\text{in.}$$

Check stirrup ratio with minimum allowed transverse reinforcement ratio per AASHTO 5.8.2.5:

$$\left(\frac{A_v}{s}\right)_{\min} = 0.0316\sqrt{f_c'}\frac{b_v}{f_y} = 0.0316\sqrt{4}\frac{96}{60} = 0.101 \text{ in.}^2/\text{in.}$$

Use stirrup ratio = 0.212 in.^{2}/in.

Compute stirrup spacing:

Try four legs of #6 bar reinforcing (A_b = 0.44 in.^{2}):

A_v = 4 (0.44 in.^{2}) = 1.8 in.^{2}

Required spacing, s = 1.8/0.212 = 8.49 in.

Check maximum spacing requirement (AASHTO 5.8.2.7):

$$\frac{v_u}{f_c'} = 0.05$$

$$s_{\max} = if\left(\frac{v_u}{f_c'} < 0.125, \min(18\text{in.}, 0.8d_v), \min(12 \text{ in.}, 0.4d_v)\right) = 18 \text{ in.}$$

For design, provide stirrup spacing of 8 in at the face of column.

Repeat the shear design steps as demonstrated above, for points along the drop cap length and produce a shear design chart. The design chart (Figure 12.5-42) shows the spacing requirements needed to satisfy the factored shear demand, V_u, as well as the minimum allowed transverse reinforcing ratios along the length of the bent cap.

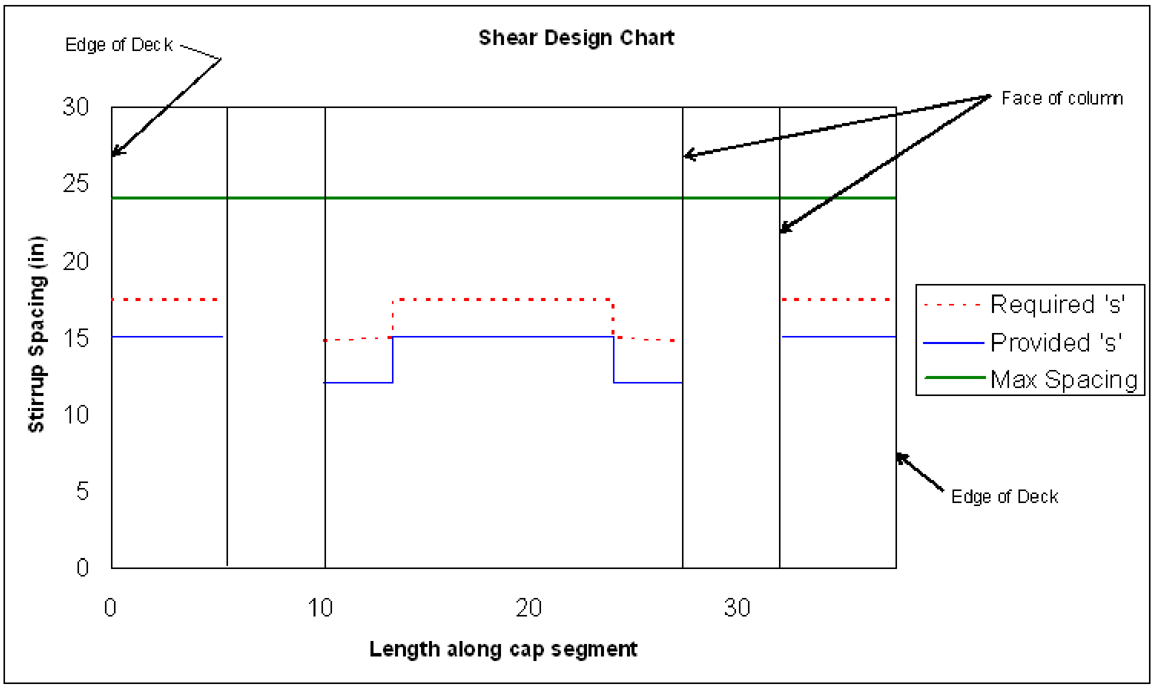


Figure 12.5-42 Shear Design Chart

12.5.2.7 Check Longitudinal Reinforcement (Shear-Flexural Interaction)

As discussed in Section 12.3.3, direct loading/support cases are typical for the drop bent cap beams. Since longitudinal reinforcements are continuous in Caltrans practice, there is no need to check shear-flexural interaction.

12.5.2.8 Detailing of Drop Cap

Additional details are discussed as follows.

12.5.2.8.1 Spacing of Longitudinal Reinforcement

The clear distance between parallel bars shall not be less than lager of 1.5(nominal diameter of bars), 1.5(maximum size of coarse aggregate) and 1.5 inches (AASHTO 5.10.3.1.1).

$s_{longit\ max}$ = 1.5 (1.27) = 1.9 in.

The following calculation determines clear spacing between the positive flexural reinforcement, accounting for clear cover and approximate outside diameters of the #6 transverse reinforcement and #10 longitudinal bars:

$$s_{longi\ provided} = \frac{7.5(12) - 2 - 2 - 0.88 - 0.88 - 24(1.44)}{23} = 2.2 \text{ in.}$$

12.5.2.8.2 Side Face Reinforcement

AASHTO 5.7.3.4 specifies that for sections that exceed three inches deep, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance d_e / 2 nearest the flexural tension reinforcing. The area of skin reinforcing, A_{sk} (in.^{2}/ft), of height on each side face shall satisfy the following equation,

$$\text{AASHTO required } A_{sk\,\min} = 0.012(d_e - 30) = 0.47 \text{ in.}^2/\text{ft}$$
(AASHTO 5.7.3.4-2)

The total area of longitudinal skin reinforcing (per face) need not exceed $A_{sk\ max}$.

A_{ps} = 0 in.^{2}

Our drop cap example does not contain prestressing.

$$A_{sk\ max} = \frac{\max\left[(A_{s\ positive})(A_{s\ negative})\right] + A_{ps}}{4} = 7.62 \text{ in.}^2$$

The maximum spacing of the skin reinforcing shall not exceed d_e / 6 or 12 in.

$$\frac{d_e}{6} = 11.6 \text{ in.}$$

Specify six #6 bars at each face. Spacing between bars will be approximately 9 inches.

SDC (SDC Equation 7.4.4.3-3) requires that the total longitudinal side face reinforcement in the bent cap shall be at least equal to $0.1A_{s\_positive}$ or $0.1A_{s\_negative}$ and shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches.

SDC Required $A_{sk\,\min} = \max\left(\left[0.1(A_{s\ positive})\right]\left[0.1(A_{s\ negative})\right]\right) = 3.05\,\text{in.}^2/\text{ft}$

Bent cap reinforcements are shown in Figures 12.5-43 to 12.5-45:

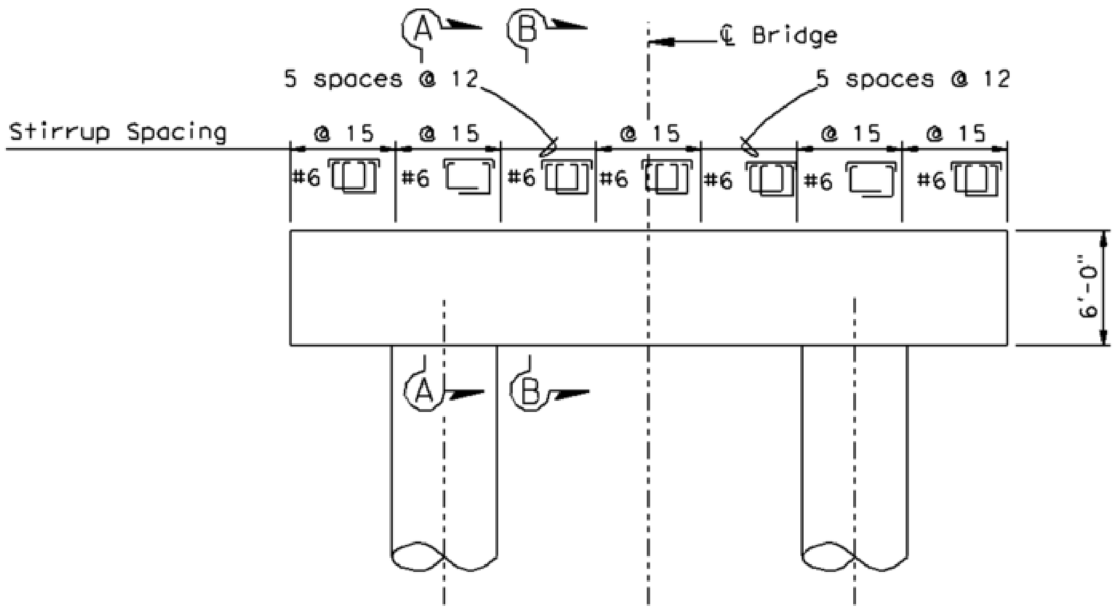


Figure 12.5-43 Elevation of Drop Cap Showing Stirrup Spacing

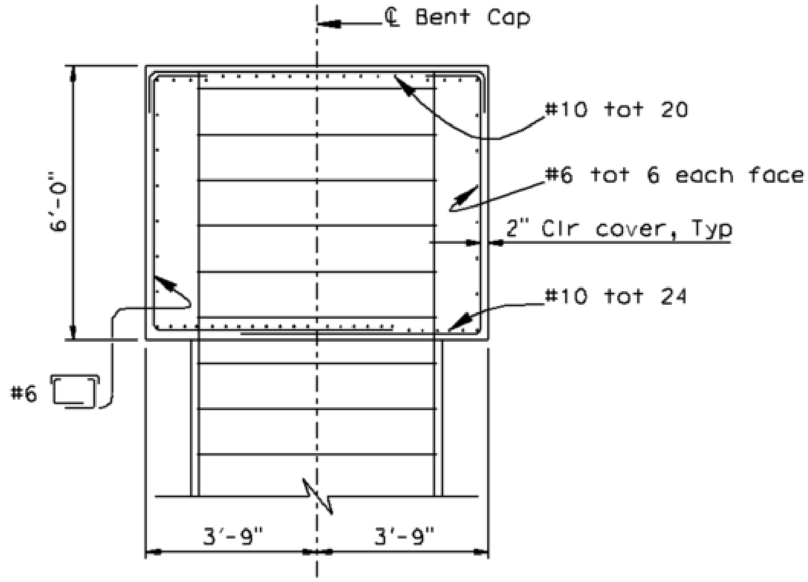


Figure 12.5-44 Section A-A

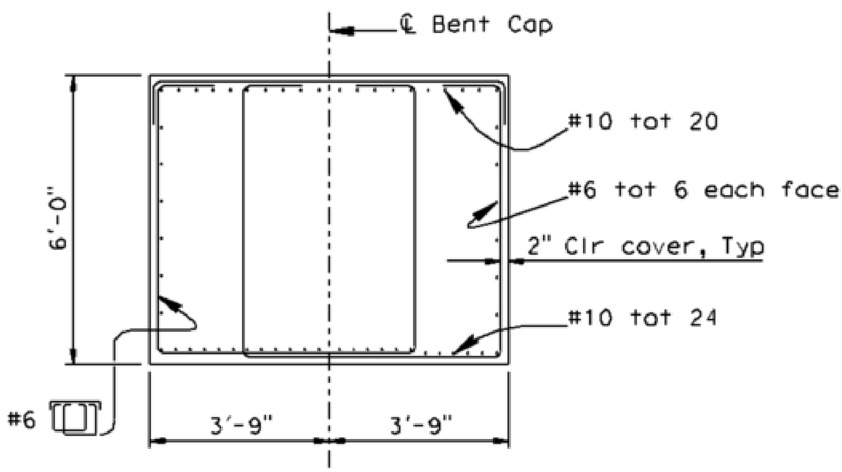


Figure 12.5-45 Section B-B

NOTATIONS

a	=	distance from the center of gravity (CG) of the superstructure to the bottom of column = height of the column + depth to CG of the bent cap (12.3.1.1.7)
a	=	$c\beta_1$; depth of the equivalent stress block (in.) (12.3.1.1.7)
A_{ps}	=	area of prestressing steel (in.^{2}) (12.3.1.1.7)
A_s	=	area of mild steel tension reinforcement (in.^{2}) (12.3.1.1.7)
A'_s	=	area of compression reinforcement (in.^{2}) (12.3.1.1.7)
A_v	=	area of shear reinforcement (in.^{2}) within a distance s (12.3.2)
b	=	top width of superstructure (12.3.1.1.7)
b	=	width of the compression face of the member (in.) (12.3.1.1.7)
b_f	=	flange width (12.1.1.3)
b_{ledge}	=	ledge width (12.1.1.3)
b_{stem}	=	stem width (12.1.1.3)
b_v	=	effective web width (in.) taken as the minimum web width within the depth d_v (12.3.2)
b_w	=	web width or diameter of a circular section (in.) (12.3.1.1.7)
c	=	distance from the extreme compression fiber to the neutral axis (12.3.1.1.5)
c	=	distance from the centerline (CL) of the columns to the edge of deck ($e + f$) (12.3.1.1.7)
d	=	distance between the CL of columns (12.3.1.1.7)
d	=	distance from the compression face to the centroid of tension reinforcement (in.) (12.1.2)
d_c	=	thickness of concrete cover measured from extreme tension fiber to center of the closest flexural reinforcement (in.) (12.3.1.2)
d_e	=	effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.) (12.3.2)
d_{ledge}	=	ledge depth (12.1.1.3)
d_p	=	distance from extreme compression fiber to the centroid of prestressing tendons (in.) (12.3.1.1.7)
d_s	=	distance from extreme compression fiber to the centroid of non−prestressed tensile reinforcement (in.) (12.3.1.1.7)
d_{stem}	=	stem depth (12.1.1.3)

d_t = distance from the extreme compression fiber to the centroid of extreme tension steel (12.3.1.1.5)

d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.) (12.3.1.1.7)

d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compression force due to flexure, it need not be taken to be less than the greater of $0.9d_e$ or $0.72h$ (in.) (12.3.2.1)

e = distance from the CL of exterior girder to the edge of deck (EOD) (12.3.1.1.7)

E_s = modulus of elasticity of reinforcing bars (ksi) (12.3.2.1)

E_p = modulus of elasticity of prestressing tendons (ksi) (12.3.2.1)

f = distance from the CL of the column to the CL of the exterior girder (12.3.1.1.7)

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) not including the effects of secondary moment, at extreme fiber of section where tensile stress is caused by externally applied loads (ksi) (12.3.1.1.2)

f_{min} = minimum live load stress resulting from the Fatigue I load combination, combined with more serve stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; . positive if tension, negative if compression (ksi) (12.3.1.3)

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and surrounding concrete (ksi) (12.3.2.1)

f_{ps} = average stress in prestressing steel at nominal bending resistance (ksi) (12.3.1.1.7)

f_r = modulus of rupture of the concrete (ksi) (12.3.1.1.2)

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi) (12.3.1.1.7)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi) (12.3.1.2)

f_y = yield strength of transverse reinforcement (ksi) (12.3.2)

f'_c = specified compressive strength of concrete (ksi) at 28 days (ksi) 12.3.1.1.7)

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi) (12.3.1.1.7)

h_{cap} = bent cap depth (12.1.1.3)

h = overall thickness or depth of the component (in.) (12.3.1.2)

h_f = compression flange depth of an I or T member (in.) (12.3.1.1.7)

I_g = moment of inertial of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4}) (12.3.1.1.2)

k = constant that depends on the type of tendon (12.3.1.1.7)

$L_{v,zero}$ = distance from the point of zero shear to the face of the support (in.) (12.1.2)

M_{cr} = cracking moment (kip-in.) (12.3.1.1.2)

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.) (12.3.1.1.2)

M_{DC} = unfactored moment demand at the section from dead load of structural components and nonstructural attachments (12.3.1.1.1)

M_{DW} = unfactored moment demand at the section from dead load of wearing surfaces and utilities (12.3.1.1.1)

$M_{u(HL93)}$ = factored moment demand at the section from HL93 Vehicle (12.3.1.1.1)

$M_{u(\mathrm{P-15})}$ = factored moment demand at the section from the Permit Vehicle (12.3.1.1.1)

$|M_u|$ = absolute value of the factored moment, not to be taken less than

$|V_u - V_p| d_v$ (kip-in.) (12.3.2.1)

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip) (12.3.2.1)

s = spacing of stirrups (in.) (12.3.2)

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3}) (12.3.1.1.2)

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3}) (12.3.1.1.2)

V_n = nominal shear resistance (kip) (12.3.2)

V_p = component in the direction of the applied shear of the effective prestressing force (kip); positive if resisting the applied shear; typically zero for conventionally reinforced bent caps (12.3.2)

V_r = factored shear resistance (kip) (12.3.2)

V_u = factored shear (kip) (12.3.2.1)

Y_t = distance from the neutral axis to the extreme tension fiber (in.) (12.3.1.1.2)

α = angle of inclination (°) of transverse reinforcement to longitudinal axis; typically 90° (12.3.2)

θ = angle of inclination (°) of diagonal compressive stresses (12.3.2)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear (12.3.2)

β_1 = stress block factor (12.3.1.1.7)

ε_x = longitudinal strain at the mid-depth of member (12.3.2.1)

ε_{cu} = failure strain of concrete in compression (12.3.1.1.5)

ε_t = net tensile strain (12.3.1.1.5)

γ = load factor for Fatigue I (12.3.1.3)

γ_1 = flexural cracking variability factor (12.3.1.1.2)

γ_2 = prestress variability factor (12.3.1.1.2)

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement (12.3.1.1.2)

γ_e = exposure factor (12.3.1.2)

ϕ = resistance factor (12.3.1.1.4)

ϕ_f = resistance factor for moment (12.3.3)

ϕ_c = resistance factor for axial load (12.3.3)

ϕ_v = resistance factor for shear (12.3.3)

Δf = live load stress range (ksi) (12.3.1.3)

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